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Journal of the
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HYDRAULICS OF CIRCULATING SYSTEMS

Clifton W. Bolieau,[1] M. ASCE

SYNOPSIS

This paper discusses the factors entering into the design of condenser circulating water systems, with particular reference to those in the TVA system. These factors include determination of design flow, pump selection and setting, waterway design, air accumulation and removal, pump testing, and system operation.

General

The purpose of a condenser is to lower the back pressure on the last stage of the steam turbine to produce a greater pressure and temperature drop between throttle and discharge, thus increasing the efficiency and capacity of the plant. The reduced pressure is within an inch or two of mercury from a perfect vacuum. Of secondary importance is the conversion of the steam to water for recycling in the boiler feedwater system. A condenser is merely a large shell and tube heat exchanger with steam in the shell and cooling water in the tubes. The circulating water system provides the cooling medium and, in a large modern steam turbine generator plant, usually consists of a pumping station and the necessary conduits for conveying the water to and away from the condenser. The actual quantity of cooling water required depends upon many design considerations, important among which are the temperature of the cooling water, the amount of heat to be rejected to the condenser, and the desired condenser back pressure. The cooling water temperatures, and thus the design temperatures for the condenser, vary over wide limits, depending upon location and season. Table 1 shows the design temperatures used for all modern TVA condensers.

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TABLE I

CAPACITIES OF TVA POWER GENERATING UNITS AND CONDENSERS

Steam Plant	Unit Numbers	Capability Megawatts Per Unit	Condenser Size, Sq Ft	Condenser Cooling Water Flow, GPM	Cooling Water Design Temperature, F	Maximum Negative Pressure in Discharge Waterbox, Feet of Water
Johnsonville	1-6	125	70,000	105,500	79	17
Johnsonville	7-10	150	65,000	99,000	65	10
Widows Creek	1-4	125	70,000	107,600	79	13
Widows Creek	5-6	125	60,000	92,500	79	13
Widows Creek	7*	500	200,000	227,000	64	16
Shawnee	1-10	150	70,000	107,600	79	23
Kingston)**	1-4	150	60,000	90,000	75	9
Kingston)	5-9	200	80,000	121,400	75	10
Colbert	1-4	200	95,000	142,500	79	21
John Sevier	1-3	200	75,000	114,000	72	19
Gallatin	1-2	250	95,000	144,200	73	11
Gallatin	3-4	275	100,000	152,000	60	11
Total	47	8,150				

*World's largest single turbogenerator unit
**World's largest steam plant

There are three commonly used tube sizes—3/4, 7/8, and 1 inch. For the same size tube bundle the smaller size offers the greatest surface area for condensation but is more subject to fouling. The large size is the least efficient but used where fouling conditions are severe. To date all TVA condensers have 7/8-inch tubes. Fouling has not been a problem, and smaller tubes may be considered for any future units. Another design point affecting the economy of the system is the velocity of the cooling water in the tubes. A low velocity permits more heat to be absorbed by the water, but the temperature difference between the warmed cooling water and the cooled condensate becomes less, requiring a larger surface area in the condenser. The design velocity at all TVA steam plants is 7.0 feet per second. Where space is at a premium, as on board ship, tube velocities of 12 feet per second are common, larger quantities of water are required, the time available for the transfer of heat is reduced, and the temperature difference is greater. Erosion and cavitation at this velocity call for more frequent tube replacement. Obviously a point of no return can be reached in building too big or too small a condenser. The length of tubes is an important consideration but is often determined by space limitations. The TVA's Widows Creek Unit 7 has 40-foot tubes; those in all other TVA units are 30 feet. The longer tubes require less water for the same result but the friction loss is greater. This is a consideration when the amount of cooling water is somewhat limited. To carry this saving of water further two or more pass condensers are often used, although none is employed in the TVA system. A 2-pass condenser uses approximately half the water required for the same results in a single-pass unit but at somewhat more than twice the head loss. The 2-pass condenser lends itself readily to reversed flow so that inlet and outlet ends may be periodically reversed and any trash washed off the tube sheets.

Some appreciable control over the temperature of the cooling water is possible by the installation of skimmer walls across the pumping station inlet channel where it leaves the river or other main source of water. These are submerged baffles which hold back the warm surface water and permit only the low-level cold water to enter the inlet channel. In the TVA system skimmer walls are in successful operation at the Kingston and Gallatin Steam Plants, reducing the summer temperature of the water several degrees and fully justifying the expense of construction.(1) It can be seen that the designs of the condenser and the circulating water system are closely related problems and must be very carefully coordinated to obtain the most economical combination, otherwise the engineer will find himself designing a system for the wrong quantity of water in an incorrectly sized condenser.

Table 1 shows the power generating capacity of TVA's seven major steam plants and the related size of condensers and required cooling water flow and design temperatures. It is interesting to note that none of these plants were in operation prior to 1951 and they now include the world's largest steam plant, and under construction is the world's largest single unit. The need for these plants stemmed primarily from the Korean War and the tremendous expansion of the country's atomic defense plants at Oak Ridge and Paducah. The loads in the TVA service area are increasing at a rate of about 2 percent greater than the United States average. In February, the peak month of 1958, the demand of 9,418 megawatts was close to the then-installed capacity of 9,700 megawatts.

In all of the TVA plants the capacity of the pumping station is somewhat greater than demanded by the condenser. The difference is that required by

TABLE 2

CAPACITIES OF TVA PUMPING STATIONS

Steam Plant	Units	Pump Capacity Per Unit, GPM	Pump Capacity Per Pump, GPM	Pump Head, Feet	Pump Motor Horse-power
Johnsonville	1-6	112,000	56,000	20.5	350
Johnsonville	7-10	108,000	54,000	19.0	350
Widows Creek	1-4	112,000	56,000	19.0	350
Widows Creek	5-6	96,000	48,000	19.0	350
Widows Creek	7	246,000	82,000	22.0	600
Shawnee	1-10	112,000	56,000	35.0	600
Kingston	1-4	97,000	48,500	21.5	350
Kingston	5-9	130,000	65,000	21.0	450
Colbert	1-4	152,000	76,000	19.5	500
John Sevier	1-3	124,000	62,000	18.0	350
Gallatin	1-2	154,000	77,000	19.0	500
Gallatin	3-4	164,000	82,000	20.0	600
Total	47	5,744,000 gpm			
	or	12,800 cfs			
	or	8-1/4 BGD			

the raw water service system in the powerhouse. Pumps for this service take their suctions from the circulating water conduit ahead of the condensers. The major use of this water is for oil and hydrogen coolers which, together with other constant and intermittent uses, adds up to several thousand gallons per minute per unit. Other pumps take their suctions from the condenser discharge conduit, but these do not affect the main pumping station capacity. These are ash disposal pumps, fire service pumps, and pumps for other services in which high-temperature water is not a controlling factor. The design capacities of the TVA condenser circulating water pumping stations, either in operation or under construction, are shown in table 2.

Although completely unrelated in quality and pressure it is interesting to note that the total pumping station capacity at these seven plants is over four times the capacity of New York City's water system and is almost equal to half of all municipal water systems in the United States.

Circulating Water System

The pumps, conduits, and condenser making up the basic parts of the circulating water system require many accessory parts to make it a smoothly working, reliable part of the power plant. Among the important components are traveling screens sized to exclude material which would not pass through the condenser tubes. High-pressure water systems for the sprays for screen cleaning are imperative. Coarse racks are always necessary to protect the screens, and even floating booms for excluding large debris are common. Valves on the discharges of pumps and inlets and outlets of the condensers are needed. Gates or stoplogs at the intake and discharge structures and pumps for unwatering and filling are necessary for periodic or emergency inspection of the system. Vacuum systems, electrical controls, cranes for servicing, and indicating and alarm devices are other necessary accessories.

The simplest system is that in a powerhouse located on the bank of a water source having little water level variation. Very short conduits are needed; pumps may be directly connected to the condensers and often without valves. Nowhere in the TVA area were such sites available because of the wide fluctuations in water level, primarily due to flood control operations. These conditions dictated that the powerhouses be set well back from the water's edge where a separate structure is needed for the screens and pumps. This is not typical of TVA plants only because many other power systems find themselves in similar situations, although some still keep the pumps in the powerhouse with gravity flow from the screen house. In the TVA plants studies showed that the space taken by the pumps in the powerhouse was too costly, and the pumps are all located with the screens at the river or the end of a canal from the river. Two major differences exist in plants with separate pumping stations—some have a single conduit between the pumping station and powerhouse and a single discharge, and the other type has individual conduits for each generating unit. The single conduit system provides greater flexibility in operation in that all pumps and condensers are manifolded to the same conduit. One of many pumps out of service for any reason has little effect on the over-all efficiency of the plant. On the other hand, the three points of manifolding introduce relatively large losses of head. This system was investigated by TVA during the preliminary design stages of its first major steam plant, and it was decided to design on the unit basis. This is also typical of other features in these plants; there is only one boiler per unit

and no cross connections in the steam piping between units. With many units per plant it is considered that a more reliable plant is obtained because the whole circulating water system does not depend upon one large conduit.

Three types of conduit were used in TVA plants—concrete-lined tunnels in rock, precast reinforced concrete pipe, and cast-in-place rectangular concrete conduits. Rock tunnels were used wherever suitable rock existed because TVA finds this to be the least expensive method. Pipe was used wherever no rock or poor rock was found, and cast-in-place rectangular concrete conduits were constructed under the powerhouse base slabs to connect with reinforced concrete pipes outside the structure. The latter type was also used at one unit where the top of rock was a convenient depth below ground surface and the distance between pumping station and powerhouse relatively short. All methods have proved satisfactory but some cracks have developed in a few sections of precast reinforced concrete pipe. These were strengthened with streamlined struts to prevent further cracking with the least obstruction to the flow of water.

There are three basic types of pumps used in circulating water systems—propeller, centrifugal, and mixed flow. The propeller type is the least expensive but its identifying characteristic of a very high shutoff head and accompanying high brake horsepower at this point practically eliminates it from consideration in systems requiring valves. It was used when installed directly on the condensers in plants where only minor water level fluctuations were expected. The vertical, centrifugal, volute type installed in a dry pit with exposed shaft is the most costly for the pump, accessories, and station in which it is installed. A dry pit deep enough to cover the maximum variation in river level is expensive to build, and a valve is required on the pump suction. To offset this extra cost this type of pump has many advantages, including the following:

The pump is rigidly mounted, eliminating sway.
Pump bearings are anchored, ensuring continuous alignment.
Double suction provides hydraulic balance which, with lower inlet speed, minimizes vibration and cavitation.
Positive lubrication of bearings.
Visual inspection of bearings at all times with no dismantling of pump for bearing replacement.
Lowest shutoff head and shutoff horsepower permits use of smaller motors.
No hydraulic thrust on the motor bearing, which carried motor, shaft, and impeller load only.

Despite these many advantages TVA could not justify the extra expense of the volute-type pump, and all installations have vertical, wet pit, mixed-flow-type units for circulators. This decision has been more than justified in the eight years of operating its major steam plants since it has never had to shut down a power generating unit because of pump failure. The pumps used are all mixed-flow, enclosed-shaft, water-lubricated type with rubber or leaded bronze bearings, driven by vertical, solid-shaft, totally enclosed motors with Kingsbury-type thrust bearings. A mixed-flow pump is one in which the head is developed partly by centrifugal force and partly by the lift of the vanes on the liquid. In a centrifugal pump the brake horsepower is a minimum at zero flow, gradually increasing to a maximum near maximum flow. This characteristic is reversed in a propeller pump. The brake horsepower curve of a

mixed-flow pump is an average of these two, resulting in a very flat curve from maximum capacity back to about 20 percent of maximum capacity, from where it rises to a maximum at zero flow. Throttling the flow at any operating condition has no appreciable effect on the power input to the motor. TVA permits a 30 percent overload above name plate motor rating for the fraction of a minute during startup when the pump is operating against a closed valve. Although the totally enclosed motors used for this service have no service factor this short overload period has caused no operating difficulties.

Most of the pumps are of the pullout type, in which the outer column, discharge connection, and base are fixed, and the shaft, shaft tube, impeller, and diffusor may be pulled out for repair or replacement. At two plants where the settings were short because of low differentials in river levels, nonpullout pumps were used. In these cases the complete pump must be removed from the pit and dismantled for inspection or repair. At each station one complete pullout element, or equivalent parts for a standard pump, are kept on hand as spare parts. The pullout type can be repaired and put back in service in a fraction of the time required for the standard type. Any circulators purchased by TVA in the future will undoubtedly be of the pullout type because the extra first cost can be justified easily by the savings in maintenance costs.

For all units except one, two pumps are provided per generating unit. The design flow from two pumps supplies the design conditions for the condenser, plus that required for other minor plant uses. A secondary high-capacity flow is always specified for the low head condition with one pump in operation. This flow permits full-load operation of the generating unit and condenser but at an increased back pressure of about 3-inch Hg at summertime maximum cooling water temperature. This, of course, requires some additional fuel and a reduction in over-all plant efficiency but no drop in load. The one exception to the two pumps per unit is at Widows Creek Steam Plant Unit 7 where, because of the large flow required, three pumps are used, any two of which will permit full-load operation.

The discharges of the pumps are brought together through a single or double wye to the main condenser supply conduit. Valves are required on each pump discharge for starting, stopping, and single pump operating conditions. It is necessary to have a valve which will operate automatically in parallel with the pump, will open slowly to minimize hydraulic disturbances, and will close rapidly to prevent reversal of flow. On the first few units built by TVA hydraulic-cylinder-operated cone valves were used because they were the most reliable and suitable valves with which TVA had previous experience. They were installed in a reduced section of the discharge to reduce cost and have given excellent service. Later, however, to further reduce the cost, full-sized butterfly valves with hydraulic operators were used. By using large valves the head loss was kept equal to or less than that of the cone valve and at an appreciable savings in first cost. In all hydraulic-cylinder-operated valves the speeds of opening or closing are adjustable through wide limits by manually adjustable throttling valves in the control piping. It recently became apparent at all plants that the operators had adjusted speeds so that the average difference between opening and closing speeds was five times. It was obvious that this difference would fit the newly developed 2-speed electric motor valve operators, and these were purchased on the unit now under construction. This represents a slight saving in first cost of the valve and a big saving in accessory equipment and space. Figure 1 shows a simplified cross section of a typical pumping station as used at a TVA plant.

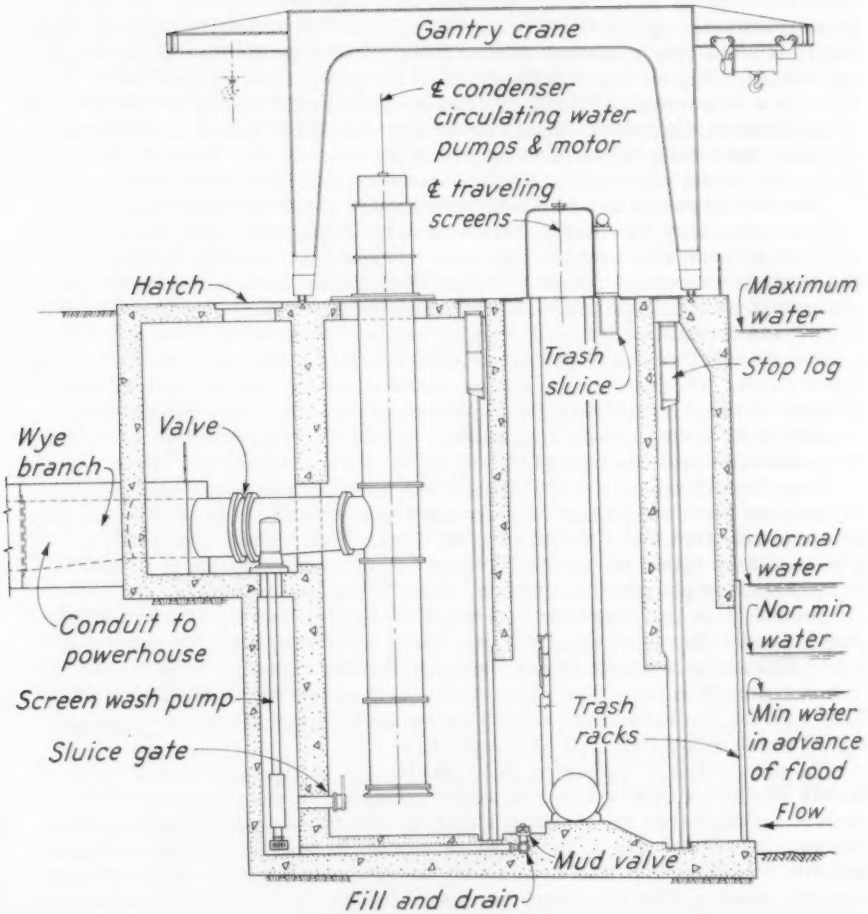

FIGURE 1

CROSS SECTION OF TYPICAL PUMPING STATION

The inlet and outlet valves on the condensers are almost all single-speed, electric-motor-operated butterfly valves. The first few units did have gate valves on the inlet but the extra floor space required was not worth the small saving in friction loss. The discharge valves are used for throttling service during tests or to control flow at those plants with varying static heads. The inlet valves are used for sectionalizing during inspection of the condenser.

Slide gates or stoplogs are provided ahead of the traveling screens at the pumping station and at the discharge structure on the outlet conduit. This permits complete dewatering of each circulating water system or, by the use of the intermediate valves on the condenser and pumps, any of several portions of the system. At all multiunit stations the systems are unwatered by

pipe connections at the conduit low points to the powerhouse drainage sump. The station drainage pumps unwater the system to be inspected into the next adjacent discharge conduit. In a 1-unit station this method cannot be used and a well is provided into the low point of the system into which a vertical turbine pump is installed when needed.

Pumping Head

Every circulating water system is different in size, steam conditions, terrain, construction costs, value of power, quality and quantity of cooling water, fluctuations in water levels, and other factors. Actual computations of one of the TVA systems would serve no useful purpose. Every designer of hydraulic structures has confidence in his own favorite methods, and the subject has been well covered by others.[2,3] With the large flows of water required it is easily demonstrated that for each power generating unit 1 foot of head requires about 50 horsepower. By capitalizing the first and operating costs of this amount of power some very substantial expenditures may be justified to save small amounts of head but these amounts will vary widely between power systems and even between individual plants in these systems. Generally, the velocities in the TVA circulating water systems have been held to about 7 feet per second in order to hold losses due to changes in velocity to a minimum. To compute the pumping head the system is divided into all of its component parts and the hydraulic losses computed as accurately as possible for each part. These losses and the velocity head are totalized and then rounded off to the next largest half foot. The steel forms used in making precast pipe and tunnel lining produces a Hazen and Williams coefficient of 130 to 140, and the losses in straight pipe or tunnel at this velocity are very low, and most of the head, external to the condenser, is found in bends, transitions, valves, exit loss, and velocity head. Each section is carefully designed to obtain the least loss of head without spending more for construction than the savings in head warranted. Many refinements are made or eliminated as dictated by the value of power savings or costs. One example of this problem is in the discharge of the system back to the river. Where pipe is used it proved uneconomical to reduce the velocity to recover some of the velocity head, and the outlets consist of the pipes discharging through headwalls at unreduced velocity. On the other hand, where tunnels are used and form work required it was considered justifiable to discharge in an increasing section and use a lesser value for exit loss. Another example is the selection of the most economical valve sizes for the pump discharges and the inlet and outlet valves on the condenser.

At Johnsonville, TVA's first major steam plant, the extra $2,200 cost each for gate valves over butterfly valves for the condenser inlet was considered justifiable to save about 1/4 velocity head in pumping costs. On subsequent plants, however, butterfly valves were used on both inlet and outlet, not because of change in pumping costs but the extra space taken by the bonnet and stem of the gate valves was considered more valuable than these pumping costs. The additional head loss produced by a lower cost small valve and the smaller but added cost of reducing and increasing conduit sections must be evaluated against the higher cost and lower losses in a large valve. The radii of bends and transitions from tunnel to the double connections on the condenser waterboxes must receive considerable attention. At plants with the

conduits in rock tunnels it was found most economical to purchase prefabricated lightweight steel wyes, tees, and reducers to act as permanent forms for the concrete rather than form these from wood.

The friction loss in the TVA systems varies between 19 and 23 feet. In three plants some static head exists in addition to the friction which increased the design heads for the pumps to somewhat more than these friction heads. About 11 feet of the friction head is in the condenser. About 2/3 of the remainder is in the supply system between the traveling screen and the condenser, and the rest is in the discharge conduit. Plants located directly on a lake or river with little difference in extreme water levels usually have circulating systems in which the head is made up entirely of friction loss because the cooling water is pumped from and to the same level. Figure 2 shows a profile and hydraulic gradient through a simplified circulating water system. One important point to note is that the discharge end of the condenser is above the gradient indicating a negative pressure. This condition can be the source of operating trouble and should be kept as low as costs will permit. The maximum negative head in the TVA system is 23 feet at Shawnee Steam Plant. This and the maximum negative heads at the other TVA plants are shown in table 1. A negative head equal to the theoretical limit has been reported for design at one plant.(4) This limit is the lowest barometric pressure anticipated less the vapor pressure of the warmest water expected. By lowering the condenser the negative head may be reduced to any partial vacuum, or even to a positive pressure, but the resulting cost of the extra excavation, basement structure, and provision for uplift soon reaches an economic limit.

Plants with long intake or discharge canals will have a measurable channel loss to add at low water levels which is flooded out at high levels. The resulting decrease in pump head will increase the cooling water flow unless artificial head is added by throttling the condenser discharge valves. Usually the plant operating forces establish an arbitrary maximum tube velocity between 8 and 9 feet per second, at which it is believed no perceptible cavitation will occur, and no throttling is done unless this limit is exceeded. This results in more efficient plant operation at the expense of minor tube erosion. In the case of TVA's Shawnee Steam Plant, located on the Ohio River, extremes of river level reach 57 feet. At this plant it was necessary to introduce artificial static head in the form of a weir in the discharge channel so that at low river stages the negative head in the discharge waterbox of the condenser would not exceed acceptable limits. This resulted in a design head of 35 feet for the pumps. Figure 3 shows a simplified profile and hydraulic gradient of this system with the two means of introducing head to eliminate high vacuum conditions—static head by weir and friction head by a throttled valve. Another method which was given some thought was to introduce an additional loss of head into the discharge conduit and eliminate any static head. This required either a small-diameter discharge conduit with dangerously high velocities or a large conduit with a nozzle or throttling valve on its discharge. The long discharge channel would have had to be deepened, and what seemed at first to be obvious savings would have been used up by other expenditures.

At TVA's John Sevier Steam Plant the flow of the river is limited by the discharge from several upstream hydro plants. These plants are usually shut down over weekends or other periods of low demand or for flood control. It was therefore necessary to build a low dam across the river to store

FIGURE 2

SIMPLIFIED PROFILE AND HYDRAULIC GRADIENT THROUGH A CONDENSER CIRCULATING WATER SYSTEM

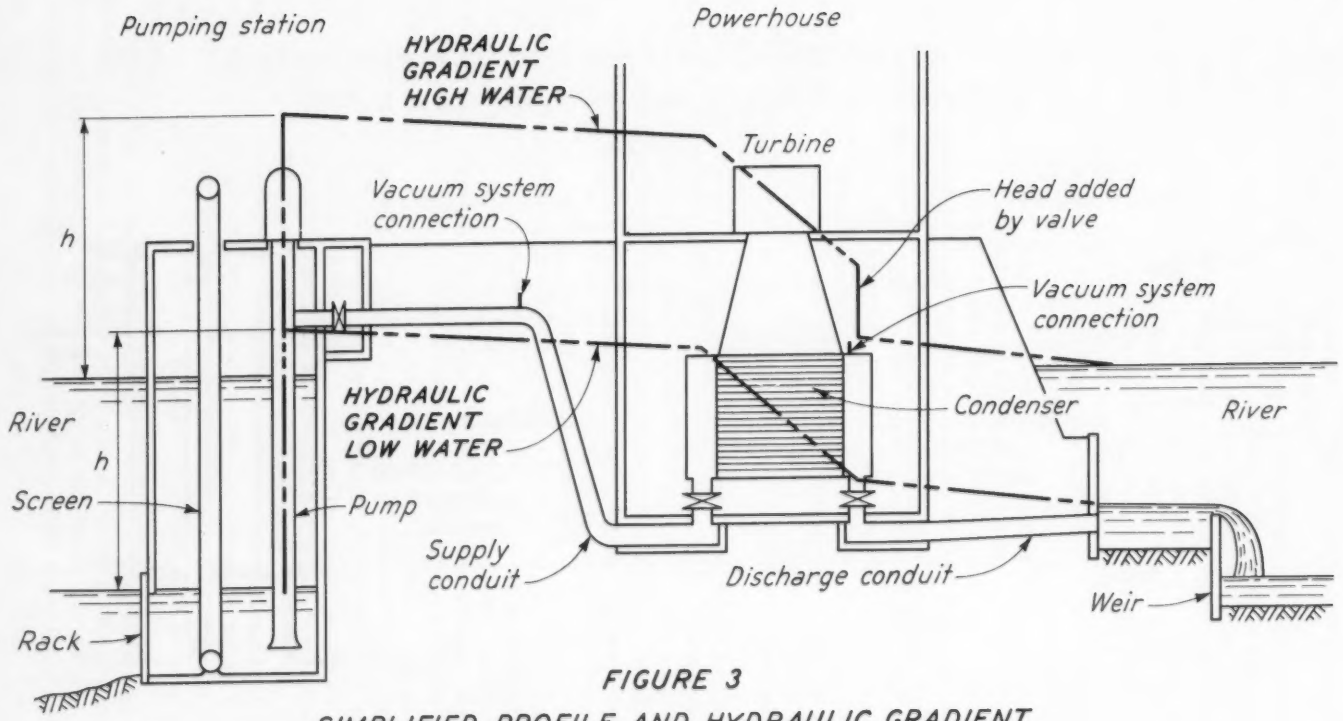

FIGURE 3

SIMPLIFIED PROFILE AND HYDRAULIC GRADIENT THROUGH A CONDENSER CIRCULATING WATER SYSTEM

enough cooling water for the steam plant for these periods. Stored water thus provides for a positive static head through the condenser circulating water system whenever the hydro plants are operating. At this time only one of the two pumps per unit is required for full flow or both pumps operate with condenser discharge valves throttled to keep tube velocity below the cavitation limit. When the flow in the river stops the water level in back of the dam gradually drops to a point where no or very little static head remains, and both pumps operate with condenser discharge valves in wide-open position. Between the two extremes the condenser discharge valves must be adjusted to maintain proper tube velocities.

Operation

Structurally the weakest parts of a circulating water system are the condenser waterboxes. In the TVA plants the test pressures of these boxes have been the shutoff heads of the pumps in that particular system during normal flood conditions. No allowance has been made for surges, and great care has been taken in the control of the pumps to prevent them. Operating instructions are prepared for each individual system, complete with a diagram showing all valves, each of which has a number. All valves regularly operated have a metal tag containing number and service for the operator's guidance. The pump discharge valves are interlocked electrically with the motors on the circulating water pumps, and the open and shut positions of these valves and those on the condenser are indicated by lamps in the control rooms. The loss of head across individual traveling screens and racks is shown by differential pressure gauges in the control room. The motors controlling the traveling screens are interlocked through pressure switches in the individual pipes to the water sprays in each screen so that no screen may operate without having the cleaning sprays in operation.

In starting up a system initially or after an extensive shutdown the system must be first filled with water. The condenser waterbox vents and those at any other high points in the system are opened, condenser and circulating water pump discharge valves closed, and water is pumped into the system by small low head pumps or by gravity from an adjacent operating system through small interconnecting piping. When the system is full the filling supply is stopped, vents closed, and the vacuum systems connected. At one plant the hydraulic gradient at minimum river level is below the level of the condenser discharge valve. In this case a vacuum connection is provided just below the discharge valve, and the discharge piping is filled by the vacuum pumps. After the complete system is full of water the condenser discharge valves are opened and the system held full of water by the vacuum system. One circulating pump is started at a time, and by electrical interlock its discharge valve starts to open. The pump comes up to speed and shutoff head in 3 or 4 seconds, but the valve does not reach open position for 90 to 120 seconds. The long column of water starts to move slowly through the throttled valve and does not reach full pump capacity for about 1-1/2 minutes. This single-pump capacity is near the secondary low head point on the pump characteristic curve. The second pump is then similarly started. It also immediately reaches shutoff head against its practically closed discharge valve. This valve slowly opens, and the flows of the two pumps are blended together in the discharge wye connection. The flow in the first pump started

will decrease as the head gradually builds up because of the increased flow, and both pumps soon operate at close to the design head.

In stopping, one pump is shut down at a time. When the motor is deenergized the discharge valve immediately starts to close by electrical interlocks between it and the pump motor. The valve closes in 20 to 30 seconds so that a minimum of reversal of flow will take place in the pump. Actual experience shows the transition from 2- to single- pump flow to be very smooth. The second pump increases in capacity during this transition to the runout point. When the second pump is stopped its discharge valve remains open and is held open by electrical interlocks. The flow in the system gradually comes to rest, and the system is held full of water by the vacuum system. When flow ceases the discharge valve on the second pump is closed manually. The system may be held full by the vacuum system or unwatered, depending upon the reason for shutdown.

In case of power failure during operation the valves are so interlocked with the motors that they will remain open so that flow in the system will gradually come to rest as in normal shutdown. Further interlocks prevent starting the pumps again unless the discharge valves are closed manually.

Pump Settings

Proper pump settings are of primary importance for efficient operation of the circulating water pumps. An improper setting invariably results in prerotation of the water before entering the pump suction or the formation of vortices on the water surface around the pump. Surface vortices cause air to be drawn into the pump suction. Prerotation of the water before it enters the suction of the pump reduces the efficiency of pumping in the same manner that a vortex does in the bottom drain of a tank. So much energy is used up spinning the water that a greatly reduced amount is left to cause flow. It is common practice in industry to install a horizontal plate over a bottom drain connection on a tank to break this vortex and greatly increase the capacity of the drain. For this reason at least one pump manufacturer recommends a very limited underclearance to prevent or cure this unseen but undoubtedly often present prerotation vortex. Another manufacturer suggests an underclearance equal to the suction bell diameter with a series of turning vanes below the suction to turn the incoming water from a horizontal to a vertical direction without rotation. It is obvious that in this system no clearance is needed between the pump and the rear wall of the pump pit. Other manufacturers suggest a deep pit with either no clearance to the back wall of the pump pit or with a baffle extending from the back wall out to the pump column. These latter two suggestions are obviously designed to stop any rotation of water around the pump column. This is a rotation caused by an improper approach channel causing more water to enter on one side than the other. It is questionable if these would have any appreciable effect on dampening a vortex formed directly under the suction bell. The TVA specifications for circulating water pumps always give the proposed dimensions of the pump pits, extreme and normal water levels, and limiting dimensions for the location of the suction bell. These limiting dimensions usually permit the manufacturer to provide a pump setting which his research has indicated will give the best performance.

Vortices drawing air into the suction bell cause reduced pump capacity and

excessive pump vibration. A set of pumps at one installation lost considerable capacity at low water where, even with 10 feet of submergence, large vortices were drawing large volumes of air into the pumps. Vibration was excessive, and although no obvious damage was done there must have been some shortening of the useful life of the bearings. In this case the vortices were stopped by hanging vertical baffles about 2 feet down into the water ahead of the pumps. Small vortices evidently formed as the water passed through the traveling screens, which at low water required the highest velocity, and then gradually grew in size as they approached the pump. Another way of stopping these same vortices was by increasing the diameter of the suction bell. This reduces the velocity of water entering the bell. Another method which the TVA has used on vertical propeller pumps, but not circulating water pumps, has been to install a floating, doughnut-shaped raft around the pump column and let it float up and down with the water. This proved very successful at one low head pumping station where it became necessary to pump water down regularly to a level much below that which the original design contemplated.

Air in System

The warmed cooling water discharging at the outlet structure invariably shows evidence of entrained air in the form of variously sized bubbles with occasional eruptions of large masses of air. Pumps for auxiliary services are often installed with their suctions connected to the circulating water discharge conduit. These sometimes lose their prime because of the accumulation of air in the suction piping. This is usually puzzling to the operator because it can occur with the net positive suction head well within allowable limits.

G. H. Van Hengel has formulated a method for determining accurately the amount of air in circulating water systems at different pressures and temperatures.(2) Inasmuch as the accuracy of results obtained by this method is not warranted for the use to which it is put, the method cited below is submitted as a simple method giving practical answers to an otherwise complex problem.

Water holds varying amounts of dissolved air, depending upon the temperature and pressure; the lower the pressure and the higher the temperature, the less air can be held in solution. Figure 4 shows this relation graphically. In the summer, river stages are usually at their lowest, water is the warmest, and the highest negative pressure can be expected in the system. Assuming a river temperature of 86 F, the cooling water will hold initially about 15 milliliters of air per liter of water at atmospheric pressure. As this water passes through the condenser it may be reduced to a negative head of 23 feet of water and the temperature increased 12 F or more. Both of these changes reduce the ability of the water to hold the air in solution, causing the water to effervesce in the same manner that carbonated water in a bottle does when the cap is removed. The solubility of carbon dioxide varies in a similar manner to air, and figure 4 will show approximately why a warm bottle of soda effervesces so much more than does a cold one.

Using the above-cited characteristics of cooling water and its changes in pressure and temperature as it passes through a condenser, the maximum probable volume of air which can be released may be computed. Water at

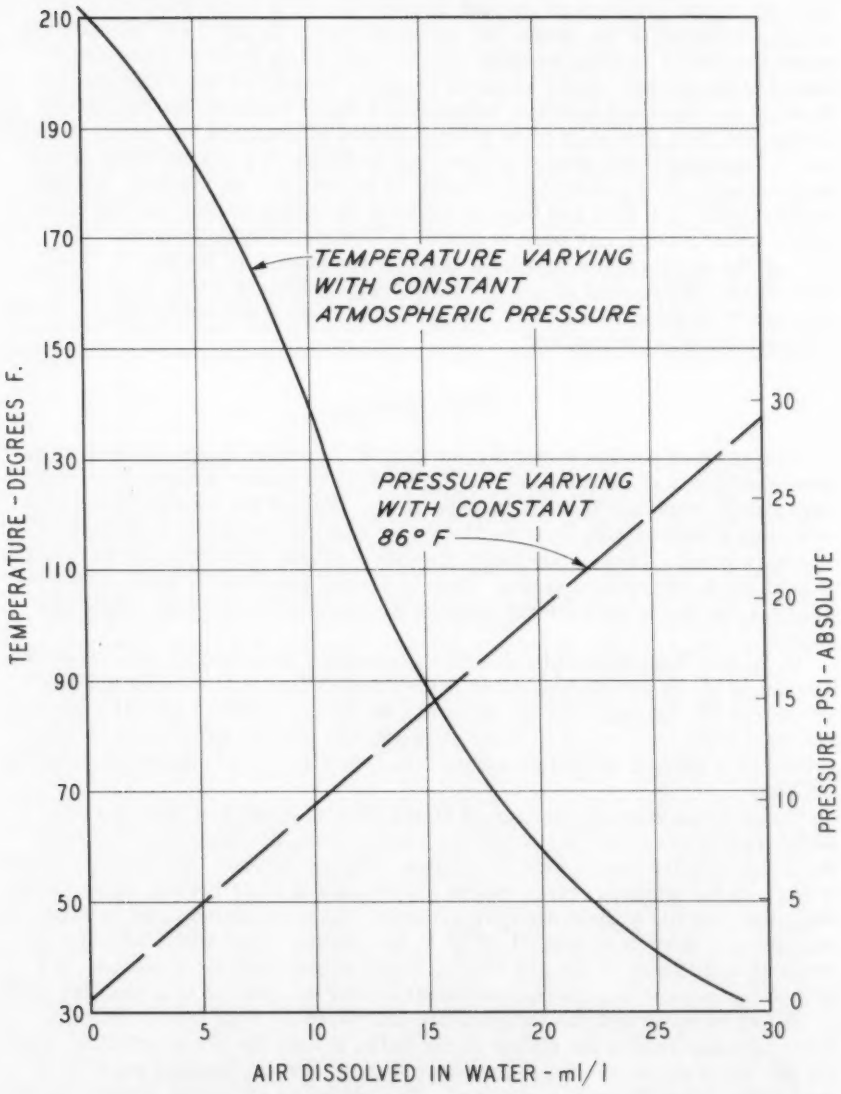

FIGURE 4
SOLUBILITY OF AIR IN WATER

atmospheric pressure and 86 F temperature may hold 15 ml/l. Water at 23 feet of water negative pressure and raised 12 degrees to 98 F temperature will hold only 3 ml/l. Converting to atmospheric pressure this indicates that 14 ml/l of air could be released. With 150,000 gallons per minute of water passing through the condenser this indicates the possible release of 280 cubic feet per minute of air at atmospheric pressure. Fortunately the efficiency of the process is far from perfect, as illustrated again by the example of the carbonated water; the gas does not all leave immediately when the cap is removed but effervesces at a decreasing rate for an appreciable time.

Not only is the efficiency of the air removal process low but the downward velocity of the water in the outlet piping of the condenser is greater than the upward velocity of the smaller released air bubbles. Many bubbles of air are thus carried down into the discharge tunnel where they rise to the surface and form masses of air above the water. The warmed water cannot possibly hold as much air as it did in its cooler state. Furthermore, the air does not go back into solution as readily as it was released because it is released equally throughout its entire volume, whereas it can only be reabsorbed at the now greatly reduced number of bubbles or at the surface under the accumulated masses of air. The released air is carried along the discharge conduit by gravity and by the attrition of the flowing water and discharged at the outlet structure. The discharge conduit should be installed on a rising grade to outlet to let gravity aid in the venting of this air.

The bubbles of air not carried down to the tunnel rise to the surface of the water in the discharge waterbox of the condenser. A vacuum system with a connection at this point must be provided to remove this air as it accumulates. The required capacity of this system has often been a matter of guesswork. All vacuum pumps are rated at intake conditions or in the above example at a negative head of 23 feet of water. The possible release of 280 cubic feet per minute of air at atmospheric pressure is equal to 840 cubic feet per minute at this partial vacuum. Fortunately a vacuum system of this capacity is not required because of the low efficiency of air removal and because much of the air is carried out with the water. The TVA has provided a system having duplicate vacuum pumps capable of removing 10 percent of the maximum probable quantity of air released as computed above. This has proved satisfactory.

Vacuum System

The vacuum pumps used at all of the TVA steam plants are of the centrifugal water-sealed type. Two pumps and a vacuum receiver are furnished for each group of units. The high point in the gravity vent pipe of the condenser discharge waterbox is connected to the vacuum receiver with flow of air controlled by a vacuum float valve. The valve float keeps the valve closed whenever water is drawn up into it and opens at other times to vent the accumulated air. The pumps are controlled by vacuum switches on the receiver which are kept at a pressure several inches below the lowest pressure anticipated in the condenser. The vacuum switches are set at different pressures, and periodically the lead pump is alternated to equalize wear. All of the TVA vacuum pumps and receivers are located on the basement floor with piping as short as conveniently possible to the condensers. With the type

pump used no harm is done if one of the float valves sticks open and water is pumped through the system. This could be absolutely prevented, and should be if a dry vacuum pump were used, by installing the vacuum pumps and receiver on an upper floor about 30 feet above the highest expected hydraulic gradient elevation in the discharge waterbox. This, of course, takes considerably more piping which is to be avoided because it is very difficult to maintain tight joints in a vacuum system, and the designer should strive for as few as possible. A second vacuum system is often needed at the pumping station when the pump discharge piping is high enough to be above the hydraulic gradient. This condition does exist at some of the TVA plants, and provision is made for venting air at the pumping station by a similar but smaller system to that provided in the powerhouse.

Although to be avoided when possible, at one TVA plant it was necessary to have a high point in the supply conduits between the pumps and the powerhouse. Instead of connecting this high point with the powerhouse vacuum system, a 1-inch tube connects it to the circulating water discharge pipe of the condenser at a point above the discharge valve. A differential pressure always exists between these two points when the unit is operating, causing flow towards the condenser discharge. This scheme has worked successfully without an additional load on the vacuum system.

Slime Control

The interior of the tubes and the discharge waterboxes of condensers provide a satisfactory environment for slime organisms, and they exist, if no provision is made to inhibit their growth, in practically all fresh water systems. The raw water of the Ohio River seems to be especially bad in this respect, and all major plants on this river have provisions for chlorinating their cooling water. The effect of slime is most noticeable as an insulating layer on the inside of the tubes, thus reducing the heat transfer coefficient and evidencing itself by an increase in back pressure on the steam side. Slime accumulations have to be very bad before their effect is noticeable as an increase in the friction loss of the cooling water flow through the tubes. In checking for slime conditions both waterboxes must be inspected because sometimes the inlet waterbox may show no evidence of slime, whereas the discharge box and the discharge ends of the tubes may contain a luxuriant growth.

The most effective means for controlling slime is by chlorination. TVA has found it necessary to chlorinate the cooling water at three of its seven modern steam plants—Shawnee, Kingston, and John Sevier. At all others provision has been made for the possible future installation of similar systems by reserving an area adjacent to the pumping station for the chlorinating equipment and placing pipe sleeves in the pumping station walls so that future piping may be added with the least possible inconvenience.

At the TVA installations chlorination is done automatically on an intermittent basis by means of a program controller. Chlorination equipment having capacity adequate to dose one unit is all that is necessary, and the program controller shifts the dose from unit to unit over the predetermined intermittent cycle. This program may be changed manually to provide a wide variation in length of dose and time between doses. An optimum program is developed for each plant; one doses each unit for an hour once a day, and

another doses each unit for a few minutes every shift. One plant finds it necessary to chlorinate at all seasons, the others only when the temperature exceeds 70 F. Fortunately it is not necessary to chlorinate continuously or equipment would be required to dose at the complete plant rate.

Testing Pumps

It has never been common practice to install meters in circulating water systems to measure the flow accurately. The high first cost for any meter accurate enough to be of any use and the operating cost for pumping against the loss of head through the meter have never seemed worthwhile. Furthermore, to the writer's knowledge, none of the manufacturers of pumps of the type used have facilities for testing the pumps in their shops. It has been common practice to build an exact scale model of the pump, test this in the manufacturer's laboratory, convert the test data to prototype performance, and adjust the efficiency of the prototype by Moody's formula as follows:

$$\frac{1-E}{1-e} = (d/D)^{1/4} \times (h/H)^{1/10}$$

Where E = Prototype efficiency
e = Model efficiency
D = Prototype impeller diameter
d = Model impeller diameter
H = Prototype head
h = Model head

Frequently there is some doubt about whether the pump actually has the capacity shown by these computations. Usually the pump manufacturer puts a little more capacity in the impeller than called for just to be positive he meets the capacity requirements of his contract. However, this requires a higher brake horsepower than anticipated and is one of the reasons for casting suspicion on the accuracy of the pump characteristic curve. This, of course, is far better than having undercapacity, and no purchaser of circulating pumps would have the manufacturers do otherwise.

Infrequently the pumps may show undercapacity by too high back pressure in the condenser and low power input to the pump motors. The ASME Power Test Code 12.2-1955, covering steam condensing apparatus, states in paragraph 151, "The rate of flow of the cooling water may be computed by the heat balance method, equating the heat given up by the steam in the condenser to that absorbed by the cooling water as follows:

$$w_s (h_s - h_c) = w_w c_p (t_2 - t_1)$$

Where w_s is the quantity of steam entering the condenser from the turbine, in pounds per hour.

h_s is the enthalpy of steam entering the condenser from the turbine, in Btu per pound.

h_c is the enthalpy of liquid from steam tables for temperature of condensate (t_c) leaving condenser in Btu per pound.

w_w is the quantity of cooling water passing through the condenser per unit of time in pounds per hour.

c_p is the specific heat of cooling water for mean temperature and for salinity measured during test in Btu per pound per F.

t_1 is the temperature of the cooling water entering the condenser in F.

t_2 is the temperature of the cooling water leaving the condenser in F."

The difficulty, of course, lies in measuring accurately the temperature of the cooling water leaving the condenser.

The several manufacturers of condensers have distinctive designs, resulting in different steam and water flow patterns. Calibrated mercury thermometers set 90 degrees apart on the condenser discharge pipes will in some designs show temperatures within 1-1/2 percent of the average. Other designs will show wide variations, and this occurred on a recent TVA test. In an effort to obtain a more accurate average a traverse of these discharge pipes was made with a calibrated thermocouple on a probe through stuffing boxes set 90 degrees apart. These showed not only stratification but appreciable turbulence by fluctuations of the needle on the galvanometer at most points in the traverse. A plotting of lines of equal temperature from this traverse showed the high-temperature isotherms concentrated on the side of the discharge pipe away from the condenser tubes. Although the flow was turbulent it would be expected that more water was flowing in the central area of the discharge pipes than along the sides where the surface friction impedes flow. All of these factors made it almost impossible to strike a reasonable average temperature leaving the condenser. By the time the cooling water has reached the point of discharge back to the river, temperature stratification has almost disappeared, and a very accurate average may be obtained. However, there is an unknown amount of heat which has been transmitted to the ground or concrete around the conduit, and there are usually several waste lines carrying small flows of hot and cold water to the conduit which affect the temperature to a small but unknown degree.

Another means of measuring flow is from the pressure drop in the condenser tubes as measured by a U-tube between the inlet and outlet waterboxes. The tubes are rolled into the tube sheet, producing an entrance and exit coefficient of very nearly 1.0. With clean tubes and using the tube losses of the Condenser Institute, a determination of flow may be made. Unfortunately both with this method and the thermodynamic method the pump manufacturer usually maintains that the test was run on the condenser and not on his pumps, and considering the number of variables and assumptions he can hardly be blamed.

A method which the TVA has used for determining the capacity of the circulating pumps is by current meter measurements in the intake channel to the pumping station. It is obvious that this is only practical when a straight, uniform, cross-section channel is available. The writer believes this to be by far the most accurate method of measurement of those mentioned. However, it is only practical under certain conditions and is not a common method with which pump manufacturers are familiar.

For testing the circulating water pumps for Unit 7 at TVA's Widows Creek Steam Plant provision has been made for a traverse of the condenser supply conduit by pitometer. This will be in a 9-foot 6-inch-square conduit with a

200-foot straight section upstream of the test section and a 70-foot straight section downstream. A description of the test section, the instruments to be used, the accuracy expected, and the acceptable limits of flow as determined by these measurements were included in the specifications and contract for these pumps, and satisfactory results by these tests will determine acceptance. Flows will be obtained for one, two, and three pumps in operation. These, together with shutoff head, will provide four points on the head capacity, power input, and efficiency curves. These tests will not be conducted until the latter part of 1960, when the system will be completed. With this test section it will be a relatively simple matter for the operating division to make periodic measurements to determine the continuing capacity of the pumps.

Wet Cooling Tower

The quantity of cooling water may be greatly reduced by using cooling towers in which the warmed cooling water is recirculated and its temperature reduced by evaporative cooling. A relatively small amount of makeup water is required to replenish that lost by evaporation, drift, and blowdown. There are no cooling towers on the TVA system because all plants are located on bodies of water adequate for all cooling water demands. Studies have been made to locate steam plants close to the mines where cooling towers would be required. These studies have all shown that locations near the main streams are more economical. In other words, in the TVA area it is cheaper to transport coal than electric power and to put up with the other inconveniences of remote locations. An estimate made six years ago for a single 200-mw unit showed that a steam plant with a cooling tower would cost $2,000,000 more than the same size plant with a conventional pumping station. Besides the money involved, the amount of level space at the right elevation for these towers is usually at a premium. In areas having insufficient water for a conventional system there is no alternative, and many are operating satisfactorily throughout the world. Another reason for using cooling towers, which will become increasingly important in the future, is to reduce the thermal pollution of streams.(5) About half of all the Btu's developed in a stream generating plant are dissipated in the condenser circulating water system. In other words, in a conventional system about half of the fuel burned serves no other purpose than warming up the body of water on which the plant is located. With power demand increasing at an accelerrating rate the circulating water may soon require evaporative cooling in many areas to make it useful and to comply with state or federal stream pollution regulations. Nuclear energy plants will make matters even worse because the lower heat rate characteristics of these plants will necessitate an even higher ratio of heat to be transferred to the cooling water.

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PASSAGE OF YOUNG FISH THROUGH TURBINES

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ABSTRACT

This paper develops the hypothesis that mortality among young fish passing through turbines is caused mainly by exposure to cavitation. Experimental procedure for subjecting fingerlings to pressures above and below atmospheric and to cavitation is described and the results tabulated. Theoretical aspects of the problem of turbine design relative to the passage of fish are surveyed.

INTRODUCTION

Recent research work has shown that dams of low or intermediate heads can be designed to permit the safe passage of young salmon over the spillways. If means were found to reduce, by a reasonable amount, the mortality rates among the fish passing through the turbines, the problem of safe passage of young fish past the dams would be almost completely solved.

For three years, the Washington Department of Fisheries, under contract with the Corps of United States Army Engineers, has conducted experiments to determine the extent of mortality to downstream migrating fingerlings and yearling salmon passing the McNary dam on the Columbia River. The spillway experiments gave an estimate of 100 per cent survival among the fish passing over the spillways.(1) The nominal head on the McNary dam is 92 feet.

In the McNary experiments the estimated survival rate among the fish passing through the turbines was 89 per cent.

In other mortality tests undertaken by the Washington Department of Fisheries at the lower Elwha dam on the Elwha River, where the head on the dam is 104 feet, the survival rate among fingerlings passing through the turbines was estimated to be 100 per cent.(2)

Note: Discussion open until July 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1947 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 1, February, 1959.

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In this paper the author discusses the mysterious reasons why 11 per cent of the fish passing through the McNary turbines were killed while at the Elwha dam, where the head is about 13 per cent higher, the fish passed through the turbines without significant losses.

Passage of Young Fish Through Turbines

It is not likely that the mortality among the young salmon passing through the McNary turbines is the result of impact or abrasion against the metal surfaces of the turbine blades. The Elwha turbines are small Francis wheels, each of 5,000 h.p., rotating at 300 r.p.m. Here, fingerlings pass through very narrow water passages, the minimum clearance between adjacent blades being only 3 inches. The McNary turbines are Kaplan, 6 bladed, adjustable-pitch propeller wheels, 23' -4" in diameter, each of 114,300 h.p., rotating at 85.7 r.p.m. The blade clearances are very much greater than those at Elwha.

In analyzing the causes of mortality among fish passing through the McNary turbines, it is of interest to ascertain, if possible, whether the law of survival of the fittest is involved or whether some of the fish are by chance exposed to more or less severe physical stresses depending on their path through the water passages relative to the metal blades.

Brett(3) discusses the general problem of physical stress in terms of discriminate and indiscriminate stress. He defines a discriminate stress as one which applies at any one time to individuals singly among a population and not to the group as a whole. An indiscriminate stress is one which applies to every member and is not discrete in its action.

A discriminate stress may result in significant mortalities among fingerling stock while the remainder may proceed seaward virtually unaffected. On the other hand, if a fish population is exposed to an indiscriminate stress which results in significant mortality, it means that the remainder are suffering though surviving and very little additional stress could have a catastrophic effect.

The author believes that the stresses resulting in mortality among fish passing through turbines are discriminate and that the 11 per cent loss in the McNary tests does not place the survivors on the brink of death. This belief is based on the hypothesis that mortality is caused mainly by exposure of some of the fish to cavitation, if by chance they pass through local regions of low pressure in a turbine. The closely spaced blades of the Elwha turbines result in a relatively large ratio of blade surface area to wheel capacity. As explained later, increasing this ratio is one of the important means of reducing cavitation phenomena in turbines.

It is to be noted, however, that in a turbine, fish are subjected to physical stresses other than those associated with cavitation. All of the fish encounter a reduction in pressure which is a function of the head on the dam. For example, fingerlings approaching the leading edges of the blades of the McNary turbines are subjected to an average pressure of about 27 p.s.i. gauge. After an interval of time of less than 1/2 second the fish pass the downstream edges of the blades where the pressure is approximately atmospheric.

Some of the fingerlings may pass through local regions of low pressure in the turbine. In this case they will be exposed to the unnatural environment of a partial vacuum. The effect of a partial vacuum on fish is an interesting subject for laboratory tests.

Experiments by Rowley[4] have shown that fish whose swim-bladders are conditioned to swallow water can withstand pressure changes of substantial amounts providing the pressure does not decrease below atmospheric.

Other experiments described in this paper show that exposure of a fish to a partial vacuum results in tensile stresses in the swim-bladder membrane caused by the expansion of gases in it. Another phenomenon which may accompany the partial vacuum is the release of dissolved nitrogen from the fluids dissolved in the fish's body to form nitrogen bubbles in the blood stream and tissues. If the pressure is lowered to the vapor pressure, vapor pockets form in the water while incipient bubbling of liquid from the internal organs of the fish occurs.

Rowley subjected rainbow trout contained in water in a lucite tube to gauge pressures up to 200 p.s.i. He then lowered the pressure abruptly to atmospheric. After the tests, he placed the fish in a trough for observation.

Rowley reports as follows: "Three trout were used in each test. Air bubbles were removed from the vessel and the pressure was increased with the hand pump at a rate approximately corresponding to that which would occur in a penstock. The pressure release was virtually instantaneous as in a turbine. The pressures and times of exposure are shown in the following table."

Pressure p.s.i. gauge	Time seconds[a]
50	6
50	12
100	12
100	12
150	18
150	36
200	24
200	48

[a] Time in seconds during which each group of fish was exposed to the pressure before abrupt reduction in pressure to atmospheric.

"The fish were immobilized while under pressure but resumed normal activity immediately after pressure release. A single trout caught against the relief valve was the only casualty. All others showed no detrimental effects after a two-week observation period."

The effects of exposing a fish to a partial vacuum are in some respects significantly different from those in Rowley's tests. Theoretical and

experimental aspects of this problem are discussed below with regard to (1) The fish's swim-bladder, (2) Caisson disease or the bends in a fish, (3) Cavitation, (4) Pressure waves encountered by fish passing through turbines, (5) Pressure and vacuum tests on fingerlings at the University of British Columbia, (6) Cavitation tests on fingerlings, (7) Turbine design relative to the passage of fish.

The Swim-bladder

The swim-bladder is an organ which adjusts the specific gravity of a fish to correspond with that of the water at different depths. The fish can vary the volume of the bladder contents by adsorption or secretion of gases through the blood vessels in the bladder wall.[5] This adjustment is made slowly as the fish swims from one depth to another.

Some fish, such as the perch, have closed swim-bladders. Others, such as the salmon, have open bladders into which the fish can gulp air from the water surface through the mouth, thence through a pneumatic duct leading to the bladder. The author has observed cohoe fingerlings behave in this manner after the bladders were deflated when the fish were exposed to a partial vacuum. A salmon, given sufficient time to be conscious of a reduction in pressure, can release air from the bladder through its mouth in order to adjust the volume of bladder contents to accommodate the pressure change.

The relationship between pressure and volume of the swim-bladder gases can be estimated by means of the gas laws. The equation of state $pv = RT$, in which p is the absolute pressure, v is the specific volume, R is the gas constant and T is the absolute temperature, is plotted for air at 50°F. in fig. 1. While this graph applies strictly only to air, it shows qualitatively the relationship between pressure and specific volume of gases (O_2, N_2, CO_2) contained in the swim-bladder.

When a fish is swimming in shallow water, the bladder gas pressure is only slightly higher than atmospheric. A sudden increase in water pressure exerted on such a fish passing downward through a penstock results in compressive stresses in its body. These are transmitted to the swim-bladder to cause a reduction in volume as is indicated in fig. 1. If the pressure is then suddenly reduced to atmospheric as the fish passes through the turbine, the bladder contents quickly expand to their original volume. However, a further reduction in pressure results in additional expansion of the bladder gases and tensile stresses in the bladder wall. If the water pressure decreases sufficiently below atmospheric and this partial vacuum is maintained for more than a very brief interval of time, the fish must release gas from the mouth of the bladder. Otherwise, the membrane is likely to rupture from the tensile stress induced in it.

If a fish passes through a local region of low pressure in a turbine it will very likely encounter a transient pressure wave. At the beginning of the wave, the pressure drops abruptly below atmospheric. The partial vacuum is maintained for only a fraction of a second. Then the pressure increases abruptly above atmospheric. In such an environment the fish will not likely receive sufficient warning of the approach of the wave of reduced pressure to permit voluntary release of air through the mouth of the bladder. The dynamic aspects of this problem indicate that the danger of rupture of the swim-bladder membrane increases with an increase in the rate of reduction in pressure and with an increase in the degree of partial vacuum to which the

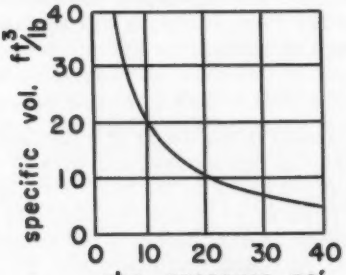


FIG. 1

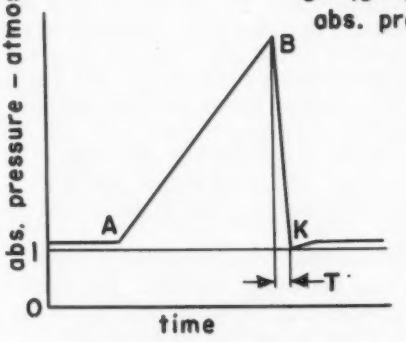


FIG. 2a

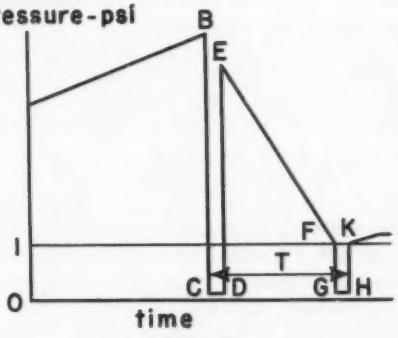
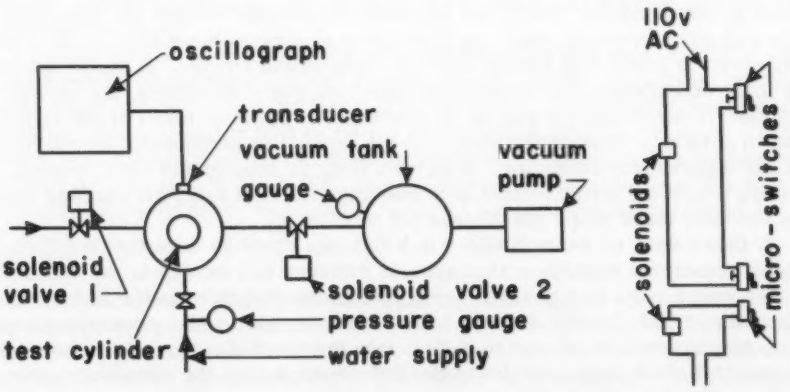


FIG. 2b



BLOCK DIAGRAM WIRING DIAGRAM

TEST APPARATUS TIME-CONTROL DEVICE

FIG. 3

fish is exposed. Whether air will be released involuntarily through the fish's mouth before the bladder wall ruptures from the tensile stress induced in it is a matter that has been investigated in tests to be described later. The author believes that few swim-bladders will rupture as a result of passage of fingerlings through turbines.

Caisson Disease or the Bends in a Fish

The conditions which result in the bends in a fish are significantly different from those which result in this disease in a diver.

The behaviour of dissolved gases in the blood stream and body fluids of a fish exposed to changes in water pressure is similar to the behaviour of dissolved gases in the reservoir water. Henry's law as applied to a reservoir may be stated as follows: At constant temperature, the maximum mass of gas which can be dissolved in a given volume of liquid is proportional to the partial pressure exerted at the liquid surface by the gas. The atmosphere contains approximately 21 per cent oxygen by volume and 79 per cent atmospheric nitrogen. The partial pressures exerted by the oxygen and nitrogen are equal to these percentages multiplied by atmospheric pressure. By substitution of the appropriate constants in Henry's law one can show that at say 50° F. the water can contain up to 11.3 p.p.m. oxygen by weight and 18.3 p.p.m. atmospheric nitrogen. (The amount of dissolved air at saturation decreases with increase in temperature.)

At 50° F., water at any elevation in a reservoir cannot normally absorb more atmospheric nitrogen and oxygen than the above amounts. The dissolved air gradually diffuses from regions of greater concentration near the reservoir surface to the depths below. Henry's law shows that no dissolved air comes out of solution from water flowing at constant temperature into a turbine and subjected to an increase in pressure or a sudden decrease in pressure to atmospheric. But if the pressure is lowered further the dissolved air will progressively come out of solution to form bubbles similar to those observed in water in a beaker when the temperature rises. If the pressure is lowered abruptly to a given value less than one atmosphere, the rate of release of dissolved air will be proportional to the total mass of air remaining in solution. This phenomenon can be expressed mathematically by means of the logarithmic decay law. It follows that the shorter the time interval during which the water pressure is less than atmospheric, the smaller will be the total mass of air that comes out of solution.

If there were no swim-bladder in a fish one could be sure that the foregoing arguments regarding atmospheric nitrogen and oxygen in the reservoir water would apply in a general way to the gases dissolved in the fish's blood stream and body fluids. But gas in the swim-bladder under pressure greater than atmospheric must tend to diffuse into the blood stream to increase the concentration of dissolved gas in the fish above that in the reservoir water surrounding it.

This is similar to the situation that applies to a diver. When he descends, the air pressure in his lungs increases above atmospheric. Some of the air in the lungs dissolves and diffuses into the blood stream and tissues to increase the concentration of dissolved gases in his body. Later, when he ascends, any decrease in water pressure results in gases, especially nitrogen coming out of solution to form bubbles in the tissues. The bubbles are carried by the blood to the lungs where they are exhaled. If the pressure change

is substantial and the decompression process takes place too rapidly, the nitrogen bubbles coalesce to form larger bubbles which may result in blocks (embolus) in the blood vessels. The stoppage of blood circulation may cause symptoms of the bends.

Scholander(6) points out that the wall of the bladder of a deep sea fish has a network of veins and arteries (rete mirabile) arranged in counter-current exchange system. He says that the rete is a bundle of small blood vessels. The veins and arteries in the bundle are arranged in such a way that the blood in one vessel flows in a direction opposite to that in an adjacent vessel. Very little gas escapes from the swim-bladder to the rest of the fish's body because the outgoing veins, highly charged with oxygen, give it up to the adjacent incoming arteries. There is a network of thousands of looping capillaries, so closely intermingled that diffusion of oxygen from veins and arteries goes on at a high rate.

The veins and arteries in the wall of the swim-bladder of a salmon are of a more embryonic character than those of the deep sea fish shown in the illustrations of Scholander's paper. Nevertheless, the veins and arteries are arranged in such a way that the blood in one vessel flows in a direction opposite to the blood in an adjacent vessel.

Oxygen absorbed from the water by the blood vessels in the fish's gills quickly enters into chemical combination in the blood stream to form carbon dioxide, an extremely soluble gas. Nitrogen being relatively more inert than oxygen dissolves in the uncombined state and comes out of solution more readily than carbon dioxide.

If it is true that the swim-bladder gases have little effect on the concentration of dissolved gases in the fish's body, then it follows that little if any nitrogen comes out of solution if the pressure on the fish is lowered to atmospheric. But if the fish is exposed to a partial vacuum it is possible for nitrogen bubbles to form in its body to cause a disease in the fish similar to the bends in a man.

It is to be expected that the severity of an attack of the bends in a fish will increase with the degree of partial vacuum and the length of the time interval during which the vacuum is applied. If a fish encounters a partial vacuum in a turbine, the time factor will be only a fraction of a second. Whether fish in general will suffer from the bends under these circumstances is a problem that was investigated in the U.B.C. tests. The author believes that few fish are likely to die from the bends as a result of passage through turbines.

Cavitation

In a turbine, some of the fish may pass through regions where cavitation may occur. In this case they will be subjected not only to the stresses associated with a partial vacuum but also to pressure intensities of high degree, resulting from the collapse of vapor pockets as the pockets and the fish are swept to regions of higher pressure in the turbine. In the U.B.C. tests, the high pressure associated with cavitation phenomena was found to be a significant factor in causing fingerling mortality.

Pressure Waves Encountered by Fish Passing through Turbines

Fig. 2a is an ideal pressure vs. time diagram showing qualitatively the variations in pressure to which fingerlings may be exposed in passing through a Francis or propeller turbine. Fingerlings subjected to this wave of

pressure would very likely have a good chance of survival. The pressure at the entrance to the penstock is shown by the ordinate at A. As the fish approach the leading edges of the turbine blades, the pressure increases to a maximum at B. After a brief interval of time T, in the order of a fraction of a second, when the fish pass the trailing edges of the blades, the pressure decreases to K.

The rate of change of pressure in the turbine, i.e. the slope of the line BK will not in practice be as uniform as that shown in the figure. The author believes that the rate of change of pressure is not a significant factor with regard to fingerling mortality: death or injury is more likely the result of exposure to cavitation.

Some of the fish may pass through local regions of low pressure in the turbine. In these local regions, the pressure-time diagram of fig. 2a may be modified as indicated in fig. 2b. (Note the change in the abscissae scale of the latter diagram.)

The fish may encounter a partial vacuum if they pass close to the leading edge of one of the blades of the turbine. This wave of low pressure is shown in fig. 2b as a trough-shaped wave to the right of B. Some fish may encounter another similar wave downstream from the trailing edges of the blades. This wave is shown to the right of F. If the pressure at the bottom of the troughs decreases to the vapor pressure of the water, vapor pockets or bubbles form in the regions of low pressure in the turbine. When the bubbles are carried farther downstream into regions of higher pressure, they collapse. Local pressure intensities of high degree then occur. These pressures would appear as a series of sharp, jagged peaks and valleys in the diagram at E and K. If the pressure at the bottom of the trough-shaped waves does not increase to the vapor pressure of the water, the vibrations at E and K would be much less pronounced.

It is well known that phenomena similar to those described above occur in some turbines for reasons to be explained later. It is of interest first to discuss experimental procedure for investigation of the effects on fingerlings of pressure waves in which the fish are exposed to a partial vacuum.

Pressure and Vacuum Tests on Fingerlings at the University of British Columbia

Several sets of experiments were undertaken in the hydraulics laboratory in 1957 and 1958. The 1957 tests were made on dead and live cohoe salmon over one year old and sockeye salmon nearly two years old, raised in the University fisheries laboratory. The 1958 tests were made on cohoe fingerlings from 1 1/2 to 4 inches long, trapped in one of the local streams. Dechlorinated Vancouver water was used in the experiments. The water temperatures varied between 49° and 57°F. In the 1957 tests and between 57° and 59° in the 1958 tests. Compressed air was discharged continuously through tubes into the trough containing the fish in order that the water supply would be saturated or nearly saturated with dissolved air. The fish were tested one at a time to observe the effects of the pressure changes.

Two types of apparatus were used for the experiments, one for subjecting the fish to pressures above and below atmospheric, the other for cavitation tests. The latter apparatus is described later.

Figs. 3 and 4 show a block diagram and photograph of the pressure and vacuum apparatus. Air release cocks, not shown in the block diagram, are

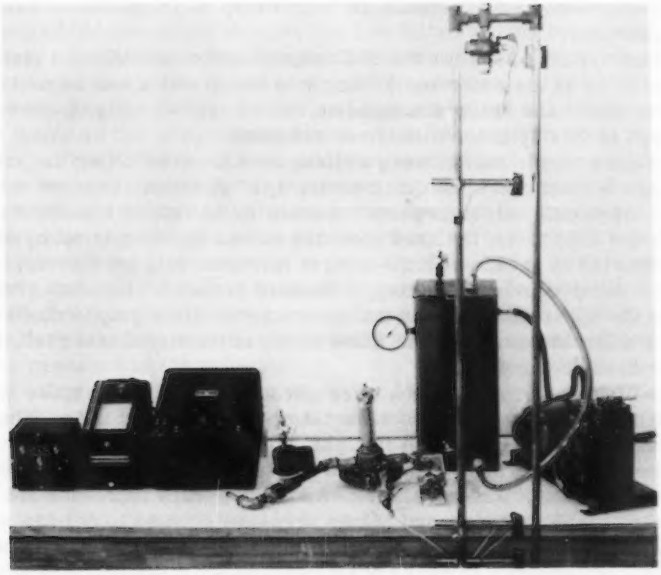


FIG. 4

FIG. 5

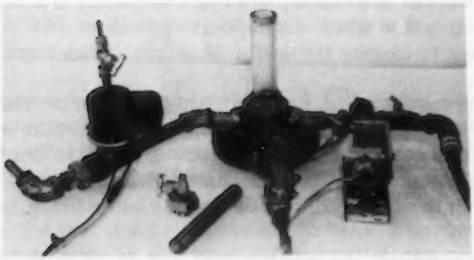


FIG. 6

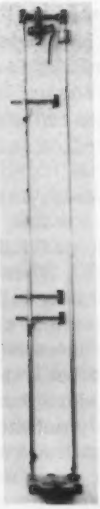


FIG. 7

provided where needed to release air which may be trapped in various parts of the apparatus.

The test cylinder used for the 1957 experiments consists of a plastic container about 4" in diameter and 8" long into which water can be poured and pressures above and below atmospheric can be applied. Fig. 5 shows a photograph of this cylinder with cover removed.

Laboratory needle valves were at first used for controlling the pressure and vacuum. Hand operated, quick-acting 1/2" globe valves were substituted later for the needle valves to permit a more rapid rate of reduction in pressure. In the 1958 tests, the hand operated valves were replaced by solenoid valves actuated by means of time-control mechanism to be described later.

After some preliminary testing, a Statham pressure transducer was connected to the apparatus in order that pressure vs. time graphs could be plotted on a Brush oscillograph. Some of the more significant preliminary tests are described below.

The bellies of four dead cohoe were slit longitudinally to expose the internal organs. The swim-bladders were clamped at the front ends. The container was filled with water, the fish placed in it and all air bubbles removed from the container. Three slow cycles of pressures were then applied between 0 and 60 p.s.i. gauge pressure. As the pressure increased the volumes of the swim-bladders decreased. Upon pressure release to atmospheric the bladders appeared to expand to their original volumes. Later, when a partial vacuum was slowly applied to the container, the swim-bladders distended and in all cases ruptured when the absolute pressure reached 7.5 to 6 p.s.i.

Three cohoe were treated similarly except that the ends of the swim-bladders were not clamped. The bladders behaved as before under positive gauge pressure. Under a partial vacuum they distended considerably before gas was progressively forced out of the bladder through the mouth of the fish. The bladders did not rupture even when the pressure approached the vapor pressure of the water. When atmospheric pressure was restored, the bladders were almost completely deflated. The bladder of one of the fish was inspected carefully under a microscope. No sign of injury could be found.

Four dead, unmutilated cohoe were placed in water in the container and a partial vacuum was slowly applied. When the absolute pressure reached 6 to 4 p.s.i. gas bubbles started to issue from the mouth or operculum of each of the fish. When the pressure approached the vapor pressure of the water, vapor pockets boiled vigorously from the mouth, operculum and anus. Some of the intestinal contents of two of the fish boiled out of the anus. The bodies became somewhat distended. In two cases, one eye was partly extruded from its socket but went back into place when atmospheric pressure was restored. The fish were then slit open to expose the swim-bladders which were almost completely deflated.

Several live cohoe were exposed to a vacuum of high degree by reducing the water pressure in the test cylinder from atmospheric to a value approaching the vapor pressure of the water. These fish were exposed to the vacuum for various periods ranging from about 10 seconds to 15 minutes. In all cases the fish died either in the test cylinder or shortly after being removed from it.

The carcasses of several fish killed by exposure to a vacuum were inspected through a bifocal microscope (magnification 20 times). In each case clusters of gas bubbles were observed in various parts of the body. These

bubbles are not necessarily evidence of an attack of the bends because the same types of bubbles were observed in fish killed simply by removing them from water. The bodies of the fish were removed from cold water and examined at room temperature and atmospheric pressure. The bubbles trapped in the tissues are the result of dissolved gases coming out of solution from the body fluids as the temperature rises. This was confirmed by cutting open the body of a fish killed by exposure to a vacuum of high degree. The fish was dissected under atmospheric pressure in cold water and observed therein through the microscope. No bubbles were found in the carcass.

The carcass was then placed in the test chamber and the microscope focussed through the plastic cover on an area of the body. A vacuum was then slowly applied. Clouds of bubbles were observed to issue from the fish's body fluids as well as from the water surrounding it. The frequency of bubble formation increased substantially when the pressure approached the vapor pressure of the water.

A live cohoe 4 1/2 inches long, weighing 14 grams, was placed in water in the test cylinder. The pressure was reduced abruptly from atmospheric to a vacuum of 16 inches hg. (equivalent to approximately 7 p.s.i. absolute pressure). The fish was exposed to the partial vacuum for a period judged to be about 4 seconds before the pressure was abruptly increased to atmospheric. The fish was then removed quickly from the test cylinder and placed in fresh water for observation.

At first the fish swam to the water surface several times to gulp air into its mouth. It then swam slowly and listlessly in a vertical position with its tail down near the bottom of the observation vessel during the period of observation. This appears to be a symptom of a ruptured swimbladder. At irregular intervals the mouth snapped wide open and shut. This is evidently one of the symptoms of the bends. The fish died within 12 hours.

Later, a microscopic examination revealed a small hole about the size of a pinhead in the deflated swim-bladder on the underside of this organ about half way between the ends. No other sign of internal or external injury was found.

A cohoe 4 inches long weighing 8 grams was exposed to the pressure test described above. After the test it swam to the water surface to gulp air into its mouth then sank to the bottom of the observation vessel where it remained inactive for some time. It exhibited symptoms of the bends and appeared to be more seriously injured than the larger fish tested previously. The fish died within 12 hours. Microscopic examination revealed no sign of injury to the swim-bladder.

It is of interest to note that in the 1958 pressure and vacuum tests on healthy cohoe, no fish was observed to swim in a vertical position for a long period of time after the tests and none was observed to have a ruptured swimbladder.

A cohoe 4 inches long, weighing 7 1/2 grams, was exposed to the same changes in pressure as were the two fish described previously, but this time the valves were manipulated in such a manner that the fish remained under the partial vacuum for a period estimated to be only a small fraction of a second.

After the test the fish appeared to behave normally during a two-week observation period. It was then suffocated, slit open and examined under the microscope. No sign of internal or external injury was found.

After these preliminary tests, the transducer and Brush oscillograph were

connected to the apparatus. Several live cohoe and sockeye salmon were subjected first to pressures up to 68 p.s.i. gauge. Then the quick opening valve was actuated abruptly to apply partial vacuums of various degrees for periods up to 2 seconds. Atmospheric pressure was then restored.

A few of these fish exhibited symptoms of the bends after removal from the test cylinder. All survived for at least 24 hours but several died a day or two later.

It was by no means evident that the fish died as a result of the pressure waves to which they had been subjected. The laboratory raised fish appeared to be unhealthy. One of the fish died in transit from the fisheries laboratory. Microscopic examination revealed a parasitic worm in its carcass.

It was therefore decided to discontinue the tests until 1958 when healthy cohoe fingerlings might become available from one of the local streams.

In the meantime the apparatus was redesigned in order to obtain pressure waves similar to that shown in fig. 11. On the right-hand side of this diagram is shown a trough shaped wave FGHK in which the pressure decreases below atmospheric by an amount v for a time interval of duration t.

Solenoid valves were installed in the apparatus to replace the hand operated valves used previously. Solenoid valve no. 1 (see block diagram, fig. 3) is a standard type as manufactured by the Pacific Control Co. Valve no. 2 is a quick opening 1/2" globe valve opened by a solenoid adapted for this service in the laboratory. The solenoids are actuated by micro-switches. These are tripped by a falling weight guided in its descent by the rod shown in the photograph of the time-control mechanism, fig. 7. The weight is released by pressing the handle of the globe valve at the top of the mechanism.

Operation of the apparatus is as follows: The test cylinder is filled with water and the fish placed in it. The pressure in the test cylinder is raised by opening the globe valve on the water supply line. This valve is then closed. The handle on the valve at the top of the time-control mechanism is pressed to reduce the pressure in the test cylinder to approximately atmospheric in about 0.35 seconds. (Pipe connections to this valve are not shown in the block diagram.) By this time the weight has dropped to close the upper, normally open, micro-switch. Power is thus applied to open solenoid valve no. 2. A partial vacuum equal to that maintained in the vacuum tank is thus applied to the test cylinder. When the falling weight reaches the second micro-switch, which is normally closed, the switch is opened to cut off the power from solenoid valve no. 2 which then closes. An instant later the weight closes the third micro-switch which results in opening solenoid valve no. 1 to break the vacuum and restore atmospheric pressure in the test cylinder.

By raising or lowering the micro-switches on the rod to which they are attached it is possible to vary the time t (fig. 11) between 0 and 0.6 seconds. Time intervals longer than 0.6 seconds are obtained by tilting the mechanism and allowing the weight to fall along the rod at an angle to the vertical.

Some difficulty was at first experienced in obtaining the wave of partial vacuum FGHK of fig. 11. Unwanted oscillations appeared in the graph to the right of K. The oscillations resulted from vibrations in the wall of the plastic test cylinder following the abrupt increase in pressure to atmospheric. The vibrations were reduced substantially by inserting a sub-floor in the test cylinder. This reduced the length of the cylinder by one-half. Two strands of wire were then wrapped tightly around the cylinder midway between the sub-floor and the lid. The wires resulted in a further reduction in vibration of the cylinder wall.

FIG. 8

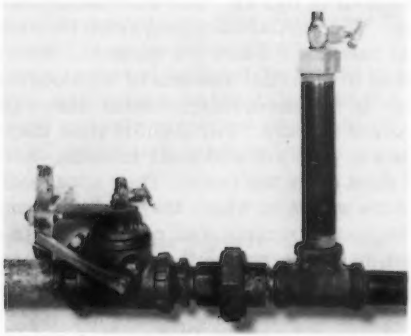


FIG. 9

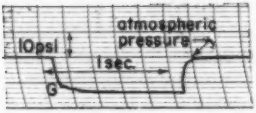


FIG. 10

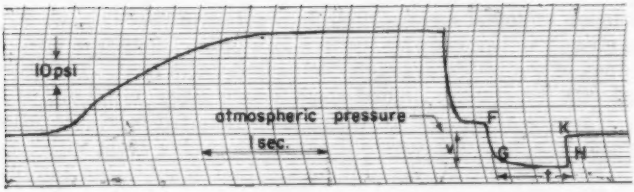


FIG. 11

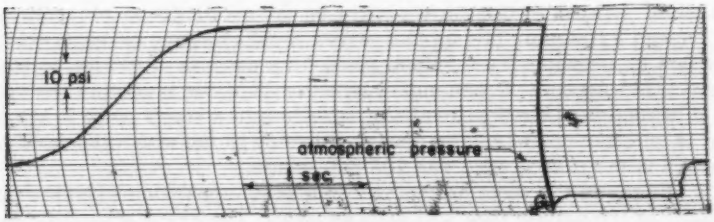


FIG. 12

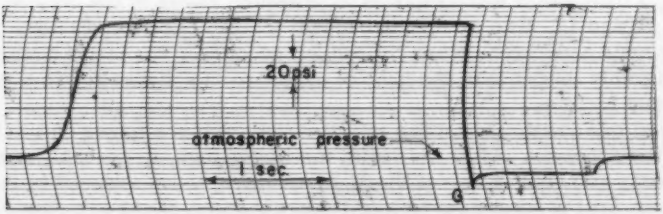


FIG. 13

Approximately one hundred fish were exposed to pressure waves of the type shown in fig. 11. The maximum pressure was approximately 68 p.s.i. gauge. The partial vacuum varied between 5" and 28.5" hg. and the time interval between 0.1 and 0.4 seconds. There were no mortalities. The fish exposed to a partial vacuum of high degree appeared to be stunned momentarily. In the observation vessel where they were placed after the tests they recovered quickly. For a short time they swam periodically to the water surface to gulp air into their mouths. All fish were kept under observation for 7 days after the tests. They apparently suffered no ill-effects from the pressure waves to which they had been exposed.

The test apparatus was re-designed in order to increase the rate of reduction in pressure and the degree of vacuum obtainable in the test cylinder. This was done to ascertain whether swim-bladders are more likely to rupture under more sharply defined pressure waves.

A plastic tube about 1" diameter and 5" long, shown in the photograph fig. 6, was substituted for the larger plastic cylinder used previously. A more rapid rate of pressure reduction was found possible in this tube than was the case in the larger cylinder.

The small cylindrical metal screen shown beside the apparatus, fig. 6, can be inserted inside the plastic tube. A fish swam readily head first from a beaker of water poured through a funnel into the tube containing the screen. After a pressure test, the fish was removed by unscrewing the lid at the top of the tube and then lifting the screen and the fish out of the apparatus.

Other changes included reconstruction of solenoid valve no. 2. A portion of the valve stem was removed by turning in a lathe. This was done to increase the area of the water passages below the valve seat. The solenoid was mounted with the plunger placed about 1 inch clear of the end of the valve stem. Closure of the micro-switch then resulted in acceleration of the plunger through this distance before it struck the valve stem with considerable force. The valve opened abruptly under the impact of the rapidly moving plunger. Fig. 8 shows a photograph of the valve and solenoid assembly.

Fig. 12 shows a typical pressure wave obtained with the reconstructed apparatus. Graphs of this wave plotted on the oscillograph recorder at a chart speed 5 times as fast as that shown in fig. 12 gave an apparent rate of reduction in pressure from 55 psi. gauge to a vacuum of 29" hg. in approximately 1/125 second. It was noted, however, that the inertia of the pen carried it beyond the true position at G, resulting in a sharp point in the graph here. A similar point is to be observed in fig. 13 for which the maximum pressure in the plastic tube was 108 psi. gauge.

Fig. 10 shows the wave plotted with the same apparatus when the initial pressure in the plastic tube was atmospheric. The graph shows the inevitable rounding of the wave at G which will occur in any apparatus of this type.

Seventy fingerlings were exposed to pressure waves of the type shown in figs. 10, 12 and 13. The maximum vacuum in each case was 29" hg. The time intervals t varied between 1.0 and 2.3 seconds. The latter time interval is considerably longer than the time of exposure of a fingerling to a partial vacuum in a turbine.

Detailed information about the effect of the pressure waves on the fish is given in the appendix. After the tests, the fingerlings, in general, were momentarily stunned. A few of them, particularly those exposed to the partial vacuum for the longer periods of time, exhibited slight symptoms of the bends. Two fish suffered from severe attacks of the bends from which they

died within a few minutes. All other fish soon recovered from the shock and in general swam to the water surface periodically for the first few minutes to gulp air into their mouths. The fish were kept under observation for 7 days. They appeared to behave in a normal manner during this time.

The conclusion drawn from these tests is that significant mortality is not likely to result from exposure of salmon fingerlings, at least those of the cohoe species tested, to a partial vacuum unaccompanied by cavitation in Francis or propeller turbines in dams of low or intermediate heads.

Cavitation Tests on Fingerlings

Fig. 14 shows a pipe rack used for water hammer experiments in the U.B.C. hydraulics laboratory. For the cavitation tests, a short length of 3/4" pipe riser and a check valve were added as shown in the figure at A and in the photograph, fig. 9. The connection for the pressure transducer is not shown in the photograph.

The cylindrical screen used previously with the plastic tube was adapted to fit inside the pipe riser. A fish swam head first into the screen when water and the fish were poured from a beaker through a funnel into the riser. The check valve was oriented to close when water flowed in the direction shown by the vector V, fig. 14. An external handle, connected to the check valve stem, permitted the check valve to be opened or closed as desired.

Operation of the apparatus for the cavitation tests is as follows: The check valve is held open to allow water to flow through the pipe rack between the two tanks. The velocity through the 500 ft. length of 3/4" pipe is 4.6 ft. per sec. When the check valve is closed abruptly, a wave of reduced pressure, starting at the check valve, sweeps downstream through the pipe. Theory and experiment show that the pressure in the pipe riser, containing the fish, decreases to the vapor pressure of the water. While this vacuum exists, the check valve is opened abruptly to admit water from the upper tank into the pipe rack and riser. The pressure of the incoming water at the check valve is approximately 22 psi. gauge. The collapse of the vapor pockets results in local pressure intensities of high degree in the pipe rack and riser.

In a preliminary test, a thin-walled plastic pipe riser was used instead of the steel pipe riser shown in fig. 9. Air trapped in the riser was removed through an air release cock at the top. Then, with water flowing in the pipe rack, the check valve was closed. Water was observed to be completely evacuated from the riser, leaving a vapor pocket here. When the check valve was then opened abruptly, water from the upper tank rushed in to fill the riser which shattered into several pieces when the vapor pocket collapsed.

In other preliminary tests, a rugged transducer, constructed in the laboratory, was used in conjunction with the steel riser shown in fig. 9. and the Brush oscillograph. This transducer, much less sensitive than the Statham instrument, withstood the pressure waves satisfactorily. Later it was decided to risk using the Statham transducer in order to obtain a more accurate plot of the transient pressure phenomena in the pipe riser.

Fig. 15 shows the plot of the first pressure wave thus obtained. The next test resulted in breaking the transducer. The other transducer was used for the remainder of the tests.

A peculiar feature of the U.B.C. transducer is the non-linearity of the pressure scale of graphs plotted on the recorder with this transducer. The

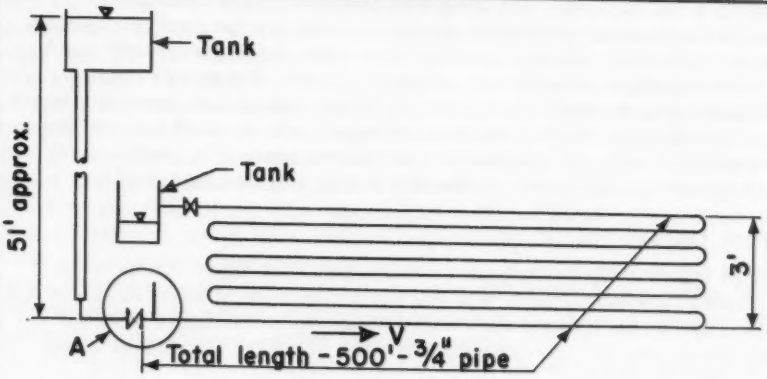


FIG. 14

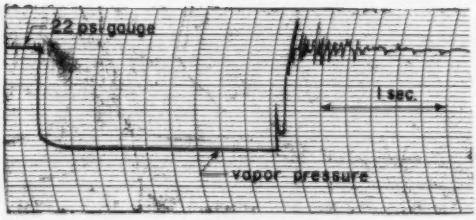


FIG. 15

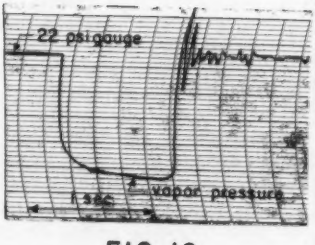


FIG. 16

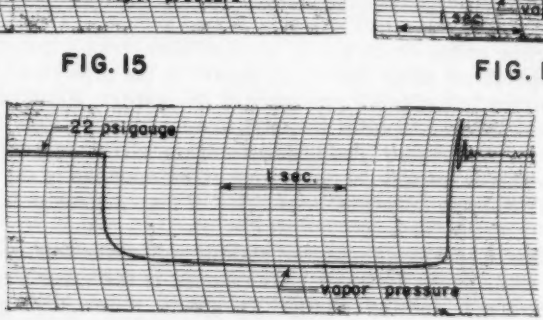


FIG. 17

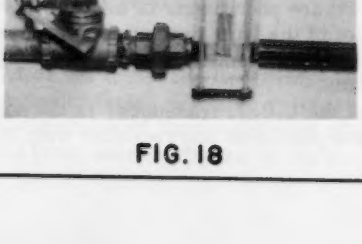


FIG. 18

ordinate scale is different for positive and negative gauge pressures. In the graphs, figs. 15, 16 and 17, the initial pressure is 22 psi. gauge. The maximum partial vacuum in each case is equal to the vapor pressure of the water. The apparatus is incapable of recording the true variations in pressure intensity accompanying the cavitation phenomena resulting from opening the check valve.

Twenty cohoe fingerlings were exposed to cavitation in the apparatus. The time of exposure to the partial vacuum was approximately 0.4 seconds. Ten fingerlings died within a short time after being removed from the pipe riser. Two more died within 4 hours afterward. Microscopic examination of the fish revealed haemorrhages in one or both eyes and in some of the gill plates. No evidence of bruises was found on the bodies. Death or injury did not appear to be selective with respect to size of the fish. Some small and some larger fish were killed while other similar fish survived. Other information about the tests is given in the appendix.

The survivors at first sank to the bottom of the observation vessel in a stunned or unconscious condition. Some fish remained upright: others lay sideways or upside down. After a few minutes they recovered sufficiently to turn upright. Some fish then swam periodically to the water surface to gulp air into their mouths until the air in their bladders was replenished. The fish were kept under observation for 7 days. At the end of this period they were apparently behaving in a normal manner.

The next set of tests involved exposing fingerlings to the same degree of partial vacuum as the twenty fish tested previously in the cavitation apparatus but this time with the high pressure intensities associated with the collapse of the vapor pocket minimized as much as possible. This was done to investigate the possibility that a partial vacuum in the pipe riser equal to the vapor pressure of the water might have more injurious effects on the fish than the maximum partial vacuum of 29" hg. obtained with the vacuum pump in other tests described previously.

In these tests, the check valve on the pipe rack was manipulated in a slightly different manner than before. While the fish was exposed to the partial vacuum, the check valve was closed gently. Fig. 17 shows a typical pressure wave thus obtained.

Ten fingerlings were subjected to the pressure wave of fig. 17. The average time of exposure to the partial vacuum was 1.6 seconds. There were no mortalities. The fish behaved in a similar manner to those subjected to a vacuum of 29" hg. in the pressure and vacuum testing apparatus described previously. They were kept under observation for 7 days and apparently suffered no ill-effects from the vacuum of high degree to which they had been exposed.

In order to observe the behaviour of fingerlings while they were exposed to a cavitation test, a thick-walled, reinforced plastic riser was constructed to replace the plastic riser that had shattered in the test described previously. A short length of cylindrical screen with the wall partly cut away was inserted in the riser. A fish could then be observed through the transparent wall of the riser. Fig. 18 shows a photograph of the riser with a fingerling in it.

Several cavitation tests were made on fingerlings in the reconstructed riser. When the check valve was opened to admit water into the vapor pocket surrounding the fish, it was noted that the fish was carried upward several inches from the bottom of the riser. Several fingerlings killed in the

apparatus were examined for signs of external injuries to the tail fin, and other parts of the body, which might have resulted from impact of the fish against the top or sides of the riser. Only one fish was observed to have a bruise on his body. It was therefore concluded that, in general, mortality resulted from mechanical shock accompanying the pressure waves that occurred when the vapor pocket collapsed in the riser.

Turbine Design Relative to the Passage of Fish

A great deal of research work has been done in recent years on cavitation in turbines and on pitting of turbine blades resulting from this phenomenon. Turbine manufacturers have sought an economic solution to the pitting problem with due regard to maintaining the efficiency and capacity of the turbines. Further research work is needed to solve the problem of safe passage of young fish through the turbines.

Experiment rather than theory is the best guide for turbine cavitation sutdies. In the laboratory, a small scale model of the turbine is placed inside a plastic casing. Cavitation phenomena are studied by visual observation or by taking photographs through the transparent casing.

Mathematical analysis of the turbine cavitation problem is difficult because of the three dimensional character of the flow. Nevertheless, qualitative conclusions can be drawn from analyses based on the Lanchester - Prandtl theory of airfoil circulation and the Thoma cavitation criterion, sigma. The author offers the following brief survey of the elementary aspects of the theory to call attention to some phases of the cavitation problem needing further study with regard to the safe passage of fish through the turbines. The discussion applies to propeller turbines but the general ideas apply equally well to Francis turbines.

Consider the forces acting on the portion of a propeller turbine blade contained in a vertical annular stream tube of unit thickness ($\Delta r = 1$) at a distance r from the shaft center. Let fig. 19 represent the development in the plane of the paper of a section through this portion of the blade. Assume that the head on the turbine and the r.p.m. are constant.

Let p be the pressure acting on a unit area of the blade where shown in the figure. The horizontal component is $p \cos\alpha$. Let P be the resultant of the water pressure acting on the upper and lower surface and P_h be the horizontal component of P. Let ds be a differential length along the surface of the blade. Then

$$P_h = \oint p \cos \alpha \, ds$$

in which $\oint$ is the line integral around the closed curve of the blade.

The torque contributed by this portion of the blade is $P_h r$ and the direction of motion in the figure is from right to left.

Now let v represent the velocity of the water relative to the blade at a distance upstream from the leading edge. Assume that v is uniform across section 1 - 1 in the diagram.

If $P \neq o$, the average unit pressure on the upper surface of the section of the blade must exceed that on the lower surface. By means of a transformation of Bernoulli's theorem into the form applying to a rotating channel[8] one can show that the average relative velocity between streamlines near the

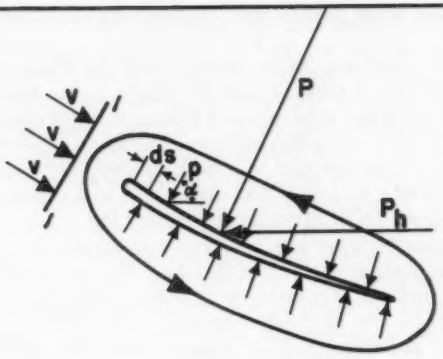


FIG. 19

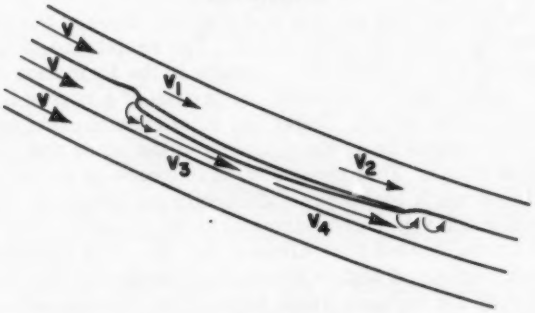


FIG. 20

FIG. 21

lower face of the blade must exceed that between streamlines near the upper face.

It follows that if an observer were moving past the blade with velocity v he would see a circulation of flow around the blade in the counter-clockwise direction of the arrows of the oval-shaped figure surrounding the blade. Each blade has a similar circulation pattern. Another observer moving with velocity v through the water channel in a path midway between adjacent blades would evidently see a circulation about him in a clockwise direction, i.e. opposite to the direction of circulation around the blade.

Figure 20 shows qualitatively the result of combining the relative velocity vectors v and the circulation pattern of fig. 19. The variation in average velocity between the various streamlines is indicated by the vectors v_1, v_2, v_3 and v_4. There will be two stagnation points - one near the leading edge of the blade, the other near the trailing edge - where the relative velocity is zero.

The disturbances in the flow pattern associated with the sudden changes in relative velocity near the leading and trailing edges of the blade result in high local shearing stresses in the water. The resistance of the water to the shearing stresses, i.e. the viscosity, results in the formation of vortices in these regions. One of the well-known properties of a vortex is the decrease in pressure toward its center. Under the most unfavourable conditions, the boundary layer separates from the surfaces of the blade and violent disturbances in the flow occur. This results in an increase in the strength of the vortices. If the strength increases sufficiently the pressure inside the vortices decreases to the vapor pressure and vapor filled cavities form as indicated by the arrow-headed part-circles, fig. 20. The cavities collapse when they reach regions of higher pressure on their way through the turbine.

Associated with the vortices which form at the leading and trailing edge are other vortices which form at the periphery of the blade. These are analogous to the vortices which occur at the tips of an aeroplane wing in flight. The tip vortices are responsible for the vapor trails which are sometimes visible behind the tips of the wings of an aeroplane flying at high altitude.

Under certain conditions, a vortex may form in the draft tube below the runner cone. When the turbine operates at best efficiency, the water leaving the trailing edges of the runner flows into the draft tube in a direction parallel to the turbine shaft, or nearly so. When the turbine is underloaded, the water whirls in the draft tube in the same direction as the rotation of the wheel. Overloads result in a whirl in the opposite direction.

It is an established practice with propeller turbines to admit air automatically through the runner cone into the draft tube to prevent rough running at low loads.[9,10] The air acts as a cushion to reduce the mechanical shock resulting from collapse of vapor bubbles formed in a violent whirl in the draft tube.

The arrow-headed part-circles in fig. 21 show the regions adjacent to a turbine blade and the region below the runner cone where cavitation is most likely to be initiated. The cavitation wakes downstream from the leading and trailing edges of each blade may be visible as spirals extending some distance into the draft tube. The whirl beneath the runner cone may extend some distance downward into the draft tube.

Cavitation phenomena in turbines can be reduced by several means. These include (1) increasing the number of blades, (2) increasing the surface area of each blade, (3) shaping the blades to minimize cavitation, (4) limiting the

maximum load carried by the turbine, (5) increasing the overall pressure in the turbine. The latter is accomplished by placing the turbine at a lower elevation with respect to the surface of the tailwater pool.

For a given horsepower of the turbine, the force P_h and the torque $P_h r$ on the section of the blade, fig. 19, can be reduced by increasing the number of blades. By this means the load is spread over a larger number of blades. This reduces the circulation about each blade and the strength of the vortices at the leading and trailing edges and at the periphery.

For a given horsepower of the turbine, increasing the length and consequently the surface area of the closed curve of the section of the blade of fig. 19 will not reduce the force P_h or the torque $P_h r$ unless the number of blades is increased. But the difference between the average unit pressure on the upper and lower face of each blade decreases as the surface area increases. It follows that the circulation decreases. The improved cavitation characteristics resulting from increasing the blade surface area are therefore similar to those resulting from increasing the number of blades.

Cavitation experiments on models of turbine runners, pump impellers and ship propellers set up in transparent casings have shown that a slight change in blade shape may result in a substantially different type of bubble formation.(7,10,12) Such model tests are an invaluable aid to the designer in choosing the most favourable blade shape to minimize cavitation.

If the horsepower of the turbine increases, the force P_h, the torque $P_h r$ and the circulation of fig. 19 increase. The most severe cavitation usually occurs when the turbine operates with fully opened gates. This points to the desirability of proper limitation of gate opening, especially when the operating head is a maximum.

The Thoma criterion used to decide on a suitable setting of the turbine with respect to tailwater elevation is called sigma (σ), a positive dimensionless number which can be calculated by means of the formula

$$\sigma = \frac{H_b - H_s}{H}$$

in which H_b is the height of the barometric water column in ft.; H_s is the elevation in ft. at which the turbine blades are placed above datum at the surface of the tailwater pool (note that H_s becomes negative if the turbine is installed below tailwater); and H is the head in ft. under which the turbine operates.

As a result of years of experience of turbine manufacturers, considerable information is available with regard to allowable values of sigma. These enable one to decide on a suitable value of H_s for propeller and Francis turbines of various specific speeds to ensure that loss in efficiency and excessive pitting of the blades from cavitation will not occur. For a dam of fixed head, H, sigma may be increased and cavitation (and the danger of injury to fingerlings) lessened by reducing H_s or, in other words, by placing the turbine at a lower elevation with respect to the surface of the tailwater pool.

Sproule and McIlquham(11) raise an interesting question with regard to the relationship between cavitation and speeds of Francis turbines. Their tests of a Francis runner showed that "less cavitation would be expected from this runner at a given power and setting, at high speed . . . . . The authors have not yet sufficient data to say whether this is a general condition

for Francis runners. Possibly the reverse would be noticed at speeds considerably above the best efficiency." Other tests by Sproule[10] on a propeller runner indicated a similar result.

In some respects, the results of these tests seem inconsistent with experience. However, among the various factors which may be involved is the circulation phenomenon of fig. 19. If the speed, i.e. the r.p.m. of the blade is increased while the horsepower is held constant, the force P_h and the torque $P_h r$ decrease. It follows that the circulation decreases.

There is a strong incentive to increase the specific speed of turbines. If a turbine is to be designed for a given head and horsepower, an increase in specific speed will result in a smaller and cheaper generator.

The specific speed of a turbine can be calculated by means of the formula

$$N_s = \frac{N(hp)^{\frac{1}{2}}}{H^{5/4}} \qquad (1)$$

in which N_S is the specific speed; N is the r.p.m.; hp is the horsepower; and H is the head in ft. under which the turbine operates.

The trend in turbine design is toward increasing the specific speed and the horsepower of each unit in order to reduce overall power plant costs. Experience has shown that one result is a more difficult cavitation problem.

A reversal of the trend toward units of larger horsepower is unattractive from the point of view of cost. Nevertheless, the tests by Sproule and McIlquham pose the following problem:

Suppose that a turbine is to be designed for specified values of N_S and H. It follows from (1) that

$$N(hp)^{\frac{1}{2}} = \text{constant}$$

For improved cavitation characteristics, should the turbine be designed to operate at a faster speed and under a lower horsepower than a turbine designed in accordance with current practice?

There are evidently a number of interesting problems for turbine designers to study with regard to the safe passage of young anadromous fish which take the route through turbines on their way to the sea. Evidence that the problems are not insoluble, at least for low head dams, is afforded by the conditions at the Elwha dam where fingerlings pass through the turbines without significant losses.

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APPENDIX

TABULATION OF TEST RESULTS

Wave type	Max. gauge pressure psi.	Max. vacuum	Time of exposure to the vacuum in seconds	Number of fingerlings tested	Length of fingerlings inches			Number of deaths	Reason for death
					max.	min.	ave.		
Fig.10	0	29" hg.	1.0	20	3.0	1.5	2.2	0	-
Fig. 12	55	29" hg.	1.0	20	3.0	1.5	2.2	0	-
Fig. 13	110	29" hg.	1.0	10	2.5	2.0	2.1	1	Bends
Fig. 12 followed by Fig. 10	55 0	29" hg. 29" hg.	1.0 1.0	10	3.5	2.3	2.4	1	Bends
Fig. 12	60	29" hg.	2.3	10	3.0	1.8	2.4	0	-
Fig. 15 (or 16)	22	vapor pressure	0.4	20	3.5	1.5	2.6	12	Cavitation
Fig. 17	22	vapor pressure	1.6	10	3.0	2.0	2.7	0	-

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Proceedings of the American Society of Civil Engineers

305,000 KW EXTENSION TO THE FISK STEAM ELECTRIC STATION

Merle H. Goedjen,[1] M. ASCE, Richard M. Collins,[2] A.M. ASCE, and John P. Roche[3]

SYNOPSIS

This paper describes the design of a large single unit extension to an historic steam electric station. Unusual features include a condenser cooling water system with a deep discharge shaft and tunnel in rock, a single concrete pedestal supporting separate high and low speed sections of a cross-compound turbine-generator and a stack meeting stringent air pollution criteria.

INTRODUCTION

The Commonwealth Edison Company system serves the city of Chicago and a large portion of northern Illinois, an area of about 11,000 square miles with an estimated population of 6,500,000. As did all the utility industry, the Company initiated a large expansion program immediately after World War II to meet its load growth. This load growth has continued and is expected to double within the next decade.

The Company's current construction program (1957 through 1961) calls for the installation of 1,605,000 kw of additional capacity which will increase its total generating capability to well over 5,000,000 kw.

In this large expansion program, the Company has had the dual problem of developing new plant sites and also expanding existing sites located near major load centers. The maximum utilization of such existing sites grows more important as new sites within economic transmission distances of load centers become progressively more difficult to obtain.

Of all the Company's plants, Fisk Station is the most strategically situated

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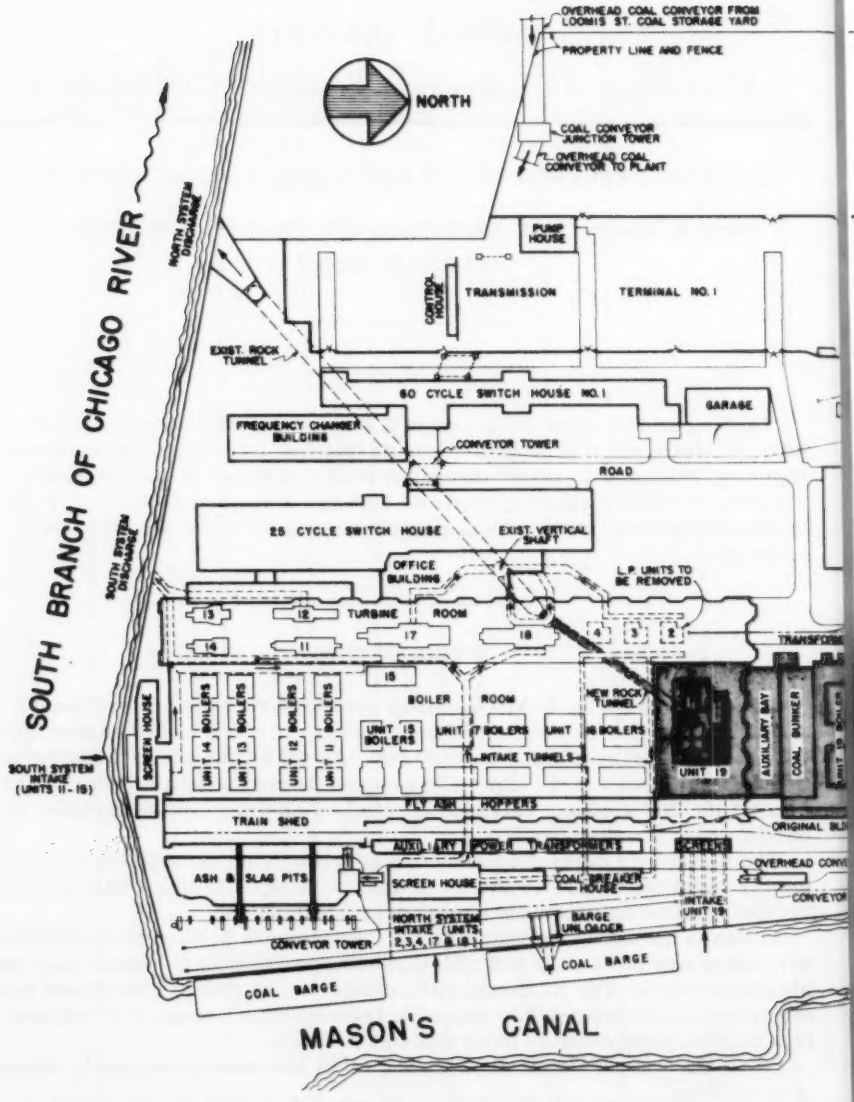

OVERHEAD COAL CONVEYOR FROM LOOMIS ST. COAL STORAGE YARD
PROPERTY LINE AND FENCE
NORTH
COAL CONVEYOR JUNCTION TOWER
OVERHEAD COAL CONVEYOR TO PLANT
SOUTH BRANCH OF CHICAGO RIVER
NORTH SYSTEM DISCHARGE
SOUTH SYSTEM DISCHARGE
PUMP HOUSE
CONTROL HOUSE
TRANSMISSION
TERMINAL NO. 1
EXIST. ROCK TUNNEL
60 CYCLE SWITCH HOUSE NO. 1
GARAGE
FREQUENCY CHANGER BUILDING
CONVEYOR TOWER
ROAD
25 CYCLE SWITCH HOUSE
EXIST. VERTICAL SHAFT
OFFICE BUILDING
L.P. UNITS TO BE REMOVED
TURBINE ROOM
BOILER ROOM
NEW ROCK TUNNEL
SCREEN HOUSE
SOUTH SYSTEM INTAKE (UNITS 11-15)
UNIT 19
AUXILIARY BAY
COAL BUNKER
TRAIN SHED
FLY ASH HOPPERS
AUXILIARY POWER TRANSFORMERS
SCREENS
ASH & SLAG PITS
SCREEN HOUSE
COAL BREAKER HOUSE
OVERHEAD CONVEYOR
NORTH SYSTEM INTAKE (UNITS 2,3,4,17 & 18)
BARGE UNLOADER
INTAKE (UNIT 19)
CONVEYOR TOWER
COAL BARGE
COAL BARGE
MASON'S CANAL

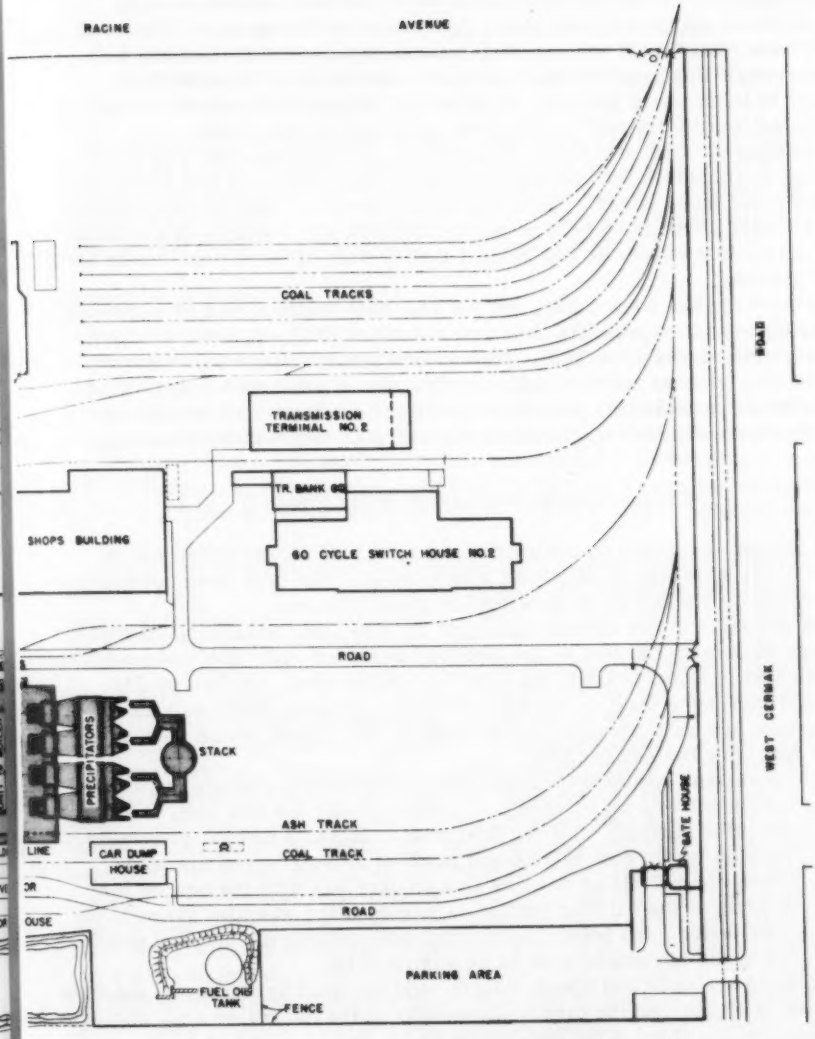


FIGURE 1

SITE PLAN

FISK STATION

COMMONWEALTH EDISON COMPANY

with respect to system load; it is located in the heart of the central section of Chicago. This plant is of considerable historic interest as well, for its original construction in 1903 represented the first central station using steam turbines exclusively for power generation in this country. The original 5000-kw (Unit No. 1) vertical turbine was returned to the General Electric Company's Schenectady plant and there erected as a "monument of courage" to those early pioneers of the utility industry who conceived and constructed this forerunner of the great machines we have today.

In planning an extension to the Fisk Station, the Company favored the installation of a single large unit because of the high efficiency and operating economy these units afford. Such an addition, however, was not practicable where existing older units were using disproportionate amounts of available space and cooling water. It was evident that certain of these units would have to be removed.

Economic studies were made, and the resultant decision was to retire the three 12,000-kw, low-pressure, vertical machines (Units 2, 3 and 4) which were originally installed in 1909. These units had not been used regularly since the war because of their high operating and maintenance costs. Their removal would provide both the added space and cooling water margin required for the new installation without the sacrifice of productive capacity.

General Design and Layout of the Plant

Fisk Station is located on the south branch of the Chicago River and is bounded on its east side by Masons Canal (Fig. 1). The total area covered by the plant and its transmission facilities is about 32 acres.

Space available in the existing plant for the new unit installation consisted of an area 60 feet by 120 feet in the turbine bay (occupied by the low-pressure units which were to be retired) and a former boiler area, 100 feet by 140 feet, which was being used as storage space. In addition, there was the area in the turbine bay once occupied by the historic No. 1 Unit, which served as laydown space for the entire plant.

The problem was how to best utilize the available space within the existing plant consistent with an economic arrangement for the new unit. Among several alternates studied: locating the cross-compound turbine-generator in the old turbine bay in both in-line and parallel close-coupled arrangements; utilizing the old boiler area as an auxiliary bay with the new boiler plant to the north of the existing station. Consideration was also given to locating the new turbine and boiler bays completely outside the existing plant and again using the old boiler area as an auxiliary bay.

All of these schemes had disadvantages, and the final arrangement selected for the new unit centered the turbine-generator in the old boiler area with its axis parallel to the front of the new boiler which was located just to the north in a new building extension. The primary advantages of this layout were:

1. It made possible a compact, close-coupled, parallel installation of the cross-compound turbine-generator with consequent savings in floor space and foundation costs since the high and low-speed sections could be mounted on a common foundation.
2. It avoided extensive remodeling and under-pinning of the existing plant turbine bay which would have been necessary due to the deck height and heavier crane requirements of the new unit.

3. It confined major demolition and remodeling work to the old boiler area resulting in minimum interference to existing plant operations.
4. It provided short, direct and economical runs of steam piping from the boiler to the turbine.
5. It permitted maximum utilization of the entire turbine bay area vacated by the removal of the low-pressure machines, as laydown space for both the existing and new units.

In addition to the demolition of the old boiler area, the layout required the removal of a portion of the adjoining ash hopper building to clear the new cooling water intake and the removal of a small structure housing the plant gas reducing station to clear the new stack. Extensive relocation of existing underground electrical cable runs was also necessary.

Equipment Arrangement

Design of the new Unit 19 plant is based on a single-boiler, single-turbine arrangement having a high cycle efficiency. The turbine-generator consists of a 3600 rpm, high and intermediate pressure turbine section, cross-compounded and close-coupled with an 1800 rpm, double-flow, condensing, low-pressure turbine section. Each turbine section is direct-connected to its own generator. The turbine throttle conditions are 2000 psig pressure, 1050 F temperature, and reheat to 1000 F with a guaranteed maximum capability of 327,000 kw when operated at one inch of mercury absolute exhaust pressure.

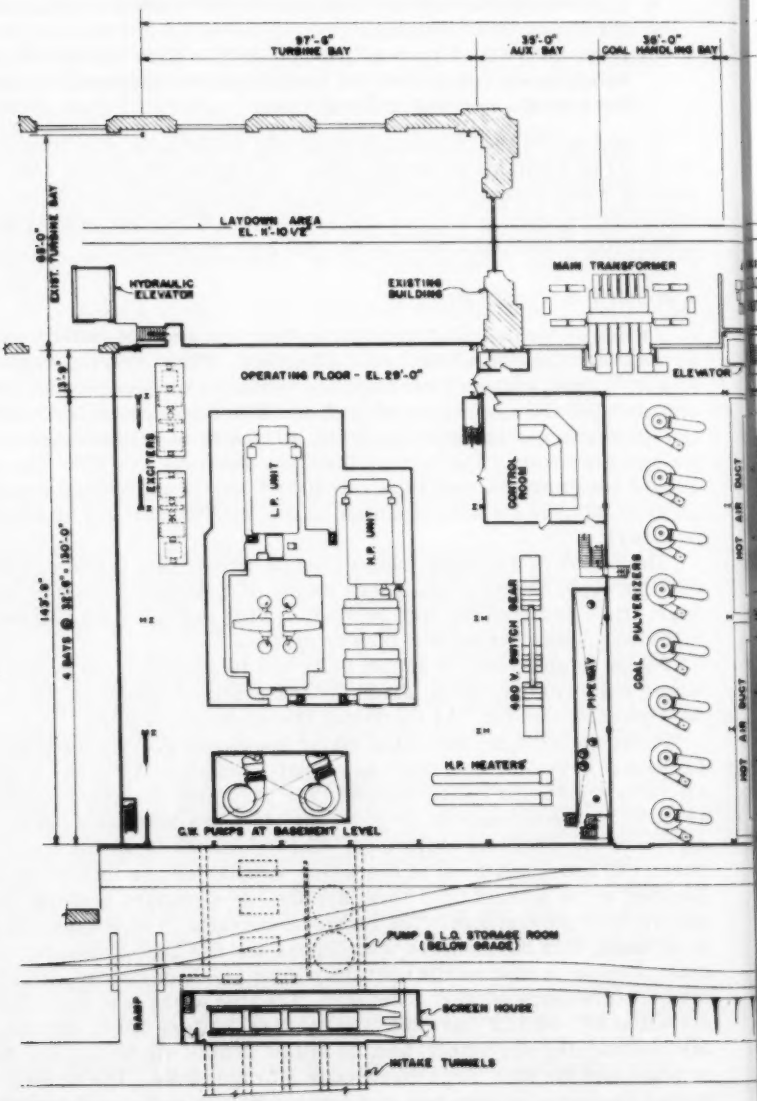
The boiler is a radiant, reheat, single-drum, twin furnace, controlled circulation type, having a continuous rating of 2,200,000 pounds of steam per hour. It is designed for both pulverized coal and gas firing and has an automatic sequential steam soot blower system.

Power is generated at 18,000 volts and is stepped up to 69,000 volts at the single main transformer. From the transformer, power is fed through underground cables into the electrical transmission system.

Physical arrangement of the major equipment divides the plant into four functional bays—turbine, auxiliary, coal-handling and boiler—with the precipitators, fans and stack located in the yard area directly behind the boiler (Fig. 2). In cross-section (Fig. 3), the plant has four main floor levels and eleven boiler platform elevations. The condenser is spring supported at the basement floor, and most of the major auxiliaries are located at this level. Located at the ground floor level are the low-pressure heaters, switchgear and the coal pulverizers. The turbine-generator is at the operating floor level along with the exciters, switchgear and the high-pressure heaters. The control room is also on the operating floor level, of course, situated opposite the high-pressure section generator. Located at the last main floor level, elevation 46' -10 1/2", are the coal feeders, battery room, chemical feed and air conditioning equipment, and, on either side of the boiler, the demineralizer plant and the four large condensate storage tanks. The deaerator is supported above the turbine bay roof level, adjacent to the coal bunker, which extends the full width of the building.

Design Criteria

Structural design of the plant is in accordance with the Chicago Building Code and its appropriate reference codes. Additional soils exploration work


97'-6" TURBINE BAY
35'-0" AUX. BAY
38'-0" COAL HANDLING BAY
LAYDOWN AREA EL. 11'-10"±
EXIST. TURBINE BAY
HYDRAULIC ELEVATOR
EXISTING BUILDING
MAIN TRANSFORMER
ELEVATOR
OPERATING FLOOR - EL. 29'-0"
EXCITERS
L.P. UNIT
H.P. UNIT
CONTROL ROOM
480 V SWITCH GEAR
PIPEWAY
COAL PULVERIZERS
HOT AIR DUCT
H.P. HEATERS
C.W. PUMPS AT BASEMENT LEVEL
PUMP & L.O. STORAGE ROOM (BELOW GRADE)
RAMP
SCREEN HOUSE
INTAKE TUNNELS

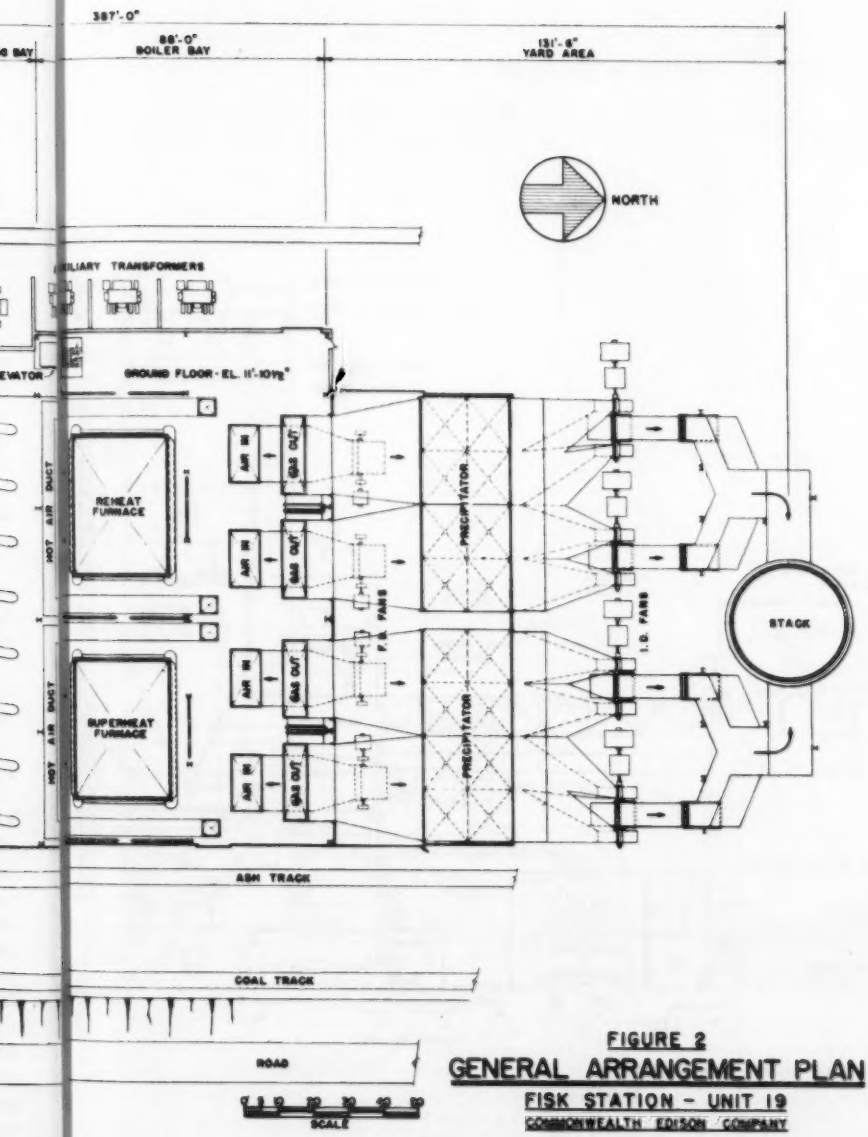
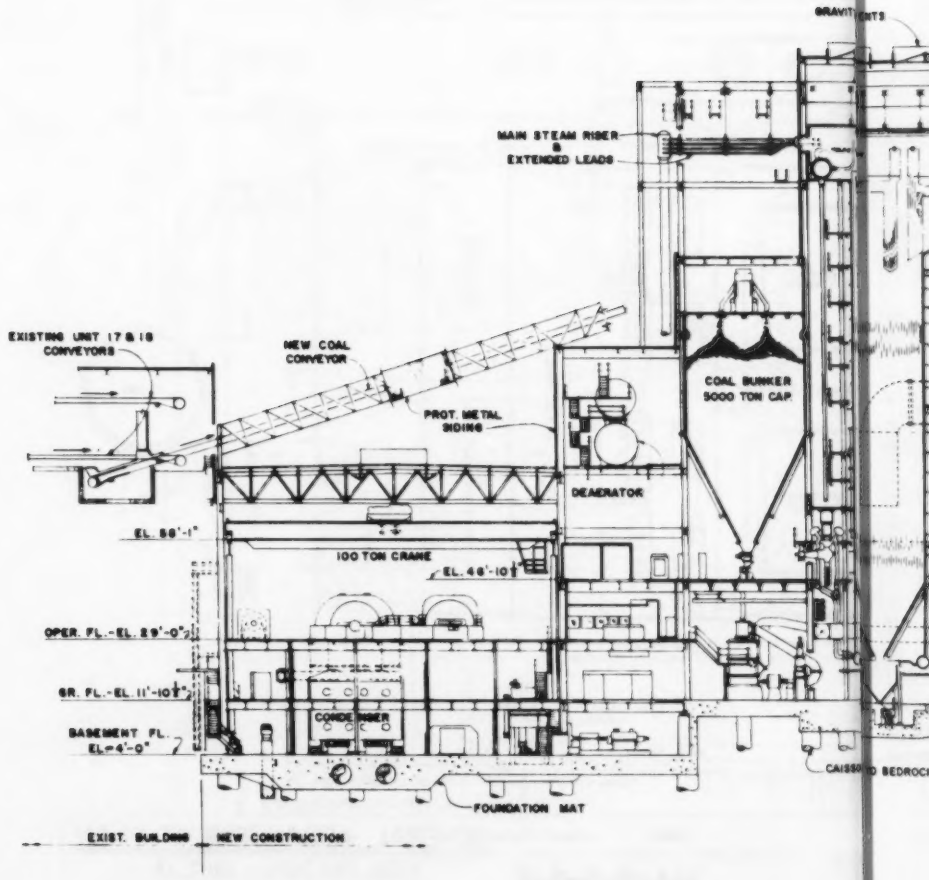


FIGURE 2

GENERAL ARRANGEMENT PLAN

FISK STATION - UNIT 19

COMMONWEALTH EDISON COMPANY


MAIN STEAM RISER & EXTENDED LEADS
EXISTING UNIT 17 & 18 CONVEYORS
NEW COAL CONVEYOR
PROT. METAL SIDING
COAL BUNKER 5000 TON CAP.
DEAERATOR
EL. 59'-1"
100 TON CRANE
EL. 46'-10"
OPER. FL.-EL. 29'-0"
GR. FL.-EL. 11'-10"
CONDENSER
BASEMENT FL. EL.-4'-0"
FOUNDATION MAT
EXIST. BUILDING
NEW CONSTRUCTION

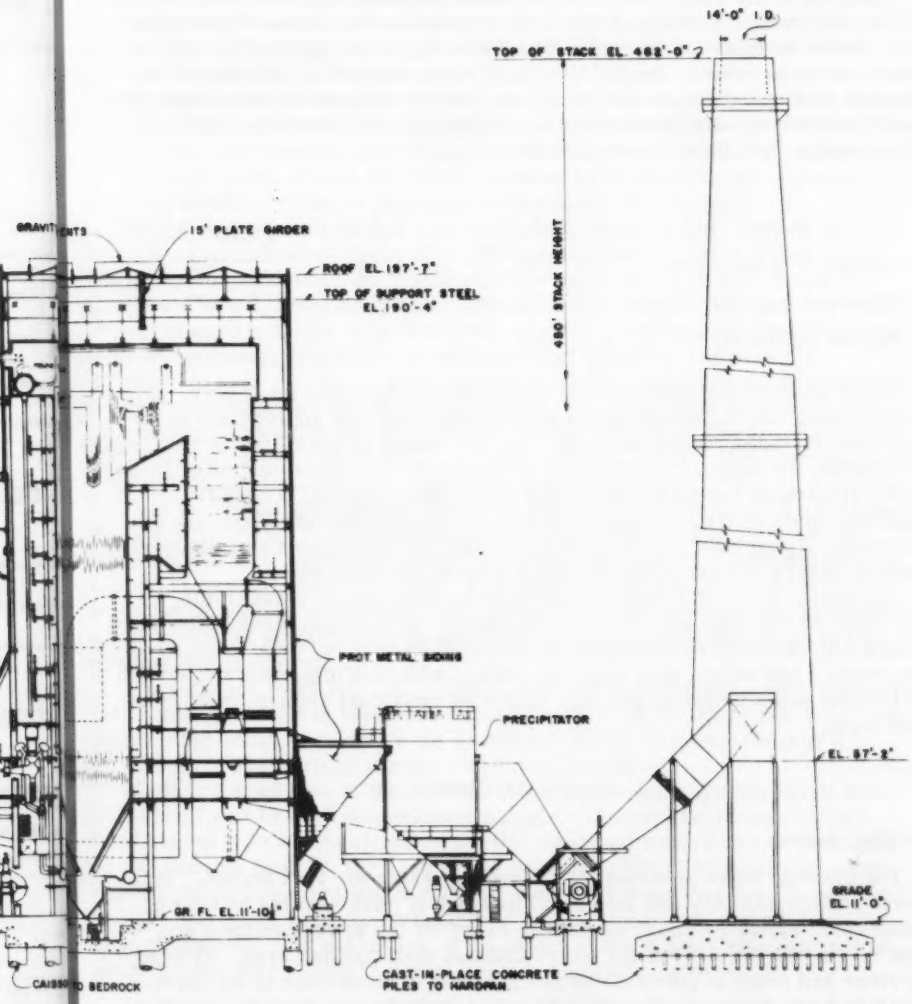


FIGURE 3

TYPICAL CROSS SECTION

FISK STATION UNIT 19

COMMONWEALTH EDISON COMPANY

0' 5 10 20 30 40 50
SCALE

was conducted to supplement existing data for the site to determine the elevations of bearing strata and their allowable bearing values.

Because of the large size of the turbine-generator and its auxiliaries, piping and conduit loads are heavier than those usually encountered in central station design. Furthermore, the Code stipulates that all floor live loads are to be posted. Design live loads were, therefore, determined as the sum of a unit piping and conduit load allowance (termed "permanent live load") and the nominal live load to be posted (termed "real live load"). These design live loads are as follows:

Area	P.L.L.	R.L.L.	Total Design L.L.
Floors - Turbine Bay	150	350	500
Floors - Auxiliary Bay	100	200	300
Boiler Platforms	-	100	100

Floor slabs and grating were designed for the real live loads only. Support framing was designed for the total design load with appropriate story and tributary area reductions in the real live loads as permitted by the Code. In addition, the support framing was designed for equipment operating loads and concentrated hanger loads for the major steam piping. These latter loads are quite heavy, ranging from 5 to 13 kips for individual hangers on horizontal piping runs and from 25 to 60 kips for the hangers supporting the vertical risers and the steam manifold adjacent to the turbine.

Equipment sizes and weights are likewise quite large: a deaerator, 49 feet long, 28 feet high with an operating weight of 450 kips; three boiler feed pumps with their hydraulic couplings and 5000 hp motors, each 8 feet by 31 feet overall and weighing 98 kips; the boiler, with plan dimensions of 88 feet by 130 feet and a height of 190 feet, imposing individual column loads up to 5200 kips.

Cooling Water System

Existing System

The Chicago River provides all the cooling water for Fisk Station. This river—and the sanitary and ship canal to which it joins—serves as both an inland waterway and an open channel to carry off the sewage effluent and most of the storm runoff of the entire Chicago metropolitan area. Flow in the river and canal is governed by two systems of locks—one at the river mouth on Lake Michigan, the other 35 miles downstream at Lockport. The purpose of these locks is to control the water level in the river and channel for navigation and, more importantly, to prevent reversal of flow of the polluted river water back into Lake Michigan, the source of Chicago's water supply.

Average water level in the river is maintained between elevations 0.0 and -2.0 feet (Chicago City Datum) as required for navigation, but maximums of +5.0 feet, during heavy storm runoff, and minimums of -5.0 feet have been recorded. For average water levels, the flow at the plant site does not exceed 1400 cfs (0.5 fps velocity) 75 percent of the time. Minimum flow has been about 800 cfs.

The cooling water system of the existing plant actually comprised two separate north and south systems (see Fig. 1). The north system, with its intake on Mason's Canal, served the old low-pressure Units 2, 3 and 4 as well as Units 17 and 18. The south system, with its intake on the river, serves Units 11 through 15. Both systems discharge into the river downstream of the south system intake.

With the decision reached to retire the three low-pressure vertical units, consideration was given to the possibility of tying the discharge of the new Unit 19 into the existing north discharge system, which would be otherwise serving only Units 17 and 18. This represented an obvious, appreciable economy over providing an independent discharge for the new unit.

The north system had been originally designed for a total flow of 397,500 gpm based on individual flows of 22,500 gpm each for Units 2, 3 and 4 and 165,000 gpm each for Units 17 and 18. The new Unit 19 cooling water requirement was 212,000 gpm, which combined with the 330,000 gpm for Units 17 and 18 gave a design flow of 542,000 gpm for the proposed integrated system. Preliminary investigation indicated that this plan was feasible.

Physical layout of the existing system (Fig. 1) consists of relatively shallow concrete tunnels from the various units, feeding into a deep vertical concrete shaft. The shaft connects to a horizontal, concrete lined, rock tunnel, 385 feet long, which terminates in another vertical shaft. This last shaft has a transition section at its top leading to the discharge structure at the river.

Four alternate schemes for tying into this existing discharge system were studied. The first two involved construction of shallow concrete tunnels from the new Unit 19 condenser to connect to the existing low-pressure unit tunnels at two different locations. The other two schemes called for construction of a vertical shaft directly below the new condenser discharge and a new tunnel (of either 9- or 13-foot diameter) to connect this shaft to the existing discharge tunnel.

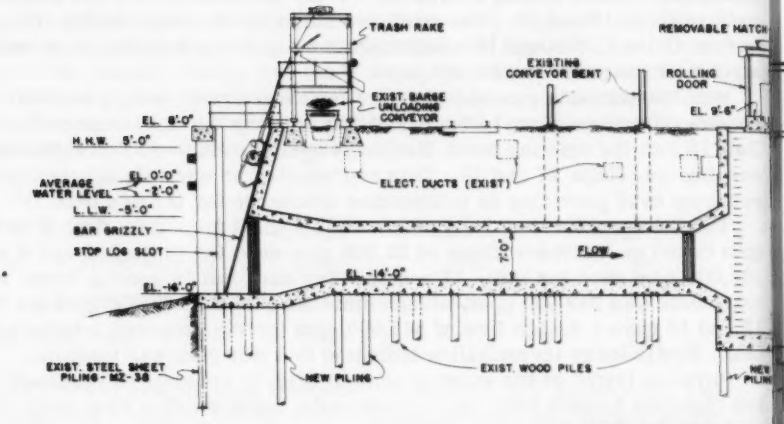
The schemes were laid out in detail, and the construction cost of each was estimated. Included in these estimates was the cost of any "down-time" on the existing units, required to tie-in the new work. A comparison of the hydraulics of the various schemes was next considered.

First, the head loss of the existing discharge system for the rated flows of Units 17 and 18 only was calculated and verified by field measurement. Head losses then were calculated for the combined rated flows of Units 17, 18 and the new Unit 19, for each of the schemes. These calculations established comparative figures on the total discharge head loss for the new unit and the increase in discharge head loss for the two existing units.

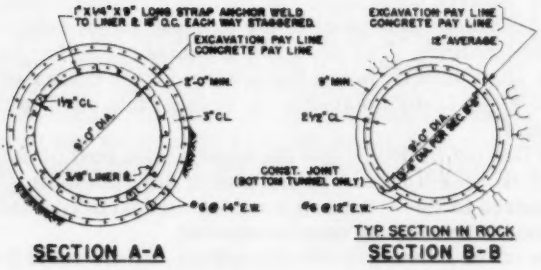
A capitalized cost evaluation of the effect of these head losses was then made for the four alternate plans, considering these factors:

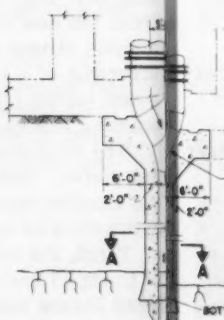
1. Total pumping head for the new Unit 19 as affecting cost of pumps and energy charges to operate them.
2. Reduced efficiencies and flows of the existing Unit 17 and 18 pumps due to increased head loss in the combined discharge system.
3. Increased energy charges to operate the Unit 17 and 18 pumps at reduced efficiencies.
4. Increase in "heat rate" or fuel consumption and also loss in kilowatt capacity of Units 17 and 18 due to increase in exhaust back-pressures as a consequence of reduced cooling water flows.

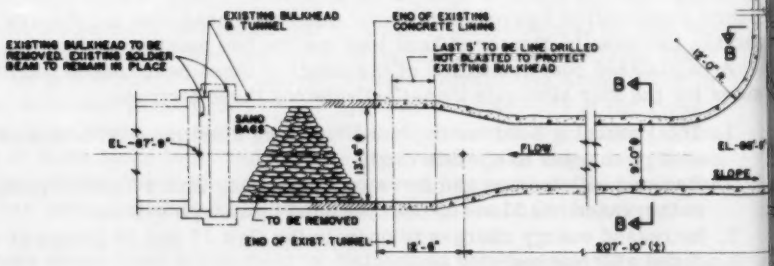
The final cost summary showed a capitalized cost savings of from



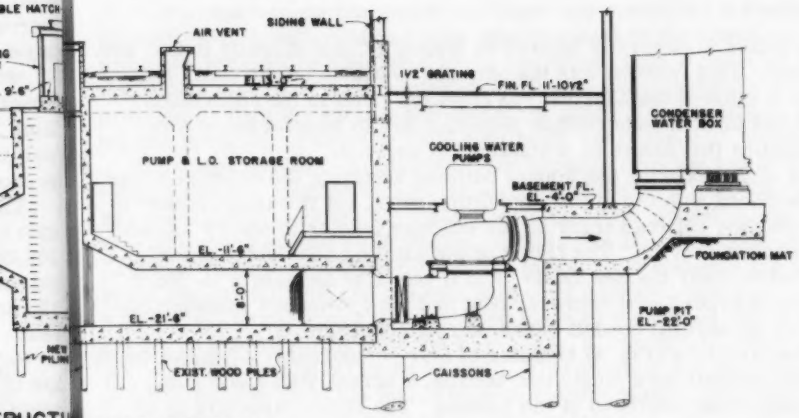
INTAKE STRUCTU CROS



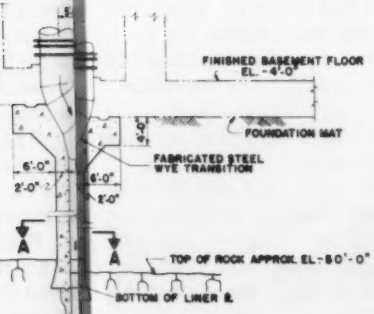


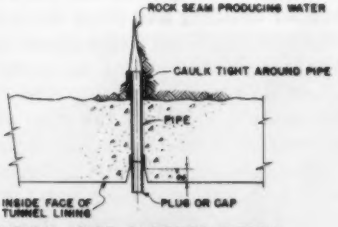


DISCHARGE SHAFT &

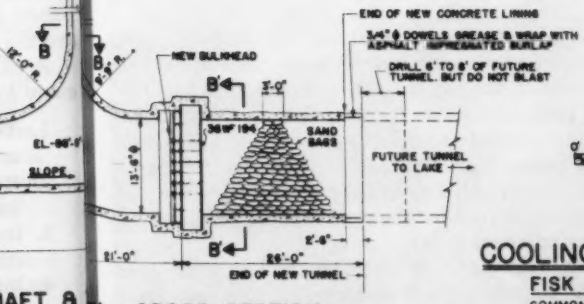


CROSS SECTION





TYPICAL PIPE BLEEDER DETAIL


END OF NEW CONCRETE LINING
¾" ϕ DOWELS GREASE & WRAP WITH ASPHALT IMPREGNATED BURLAP
DRILL 6' TO 8' OF FUTURE TUNNEL. BUT DO NOT BLAST
NEW BULKHEAD
SAND BAGS
FUTURE TUNNEL TO LAKE
SLOPE
END OF NEW TUNNEL
0' 5 10 15 20 25
SCALE

- CROSS SECTION

FIGURE 4

COOLING WATER SYSTEM

FISK STATION - UNIT 19

COMMONWEALTH EDISON COMPANY

$160,000 to $260,000 for the vertical shaft and 9 foot tunnel scheme over the other three alternates and this plan was adopted.

Intake Structure

The new intake structure is located on Mason's Canal directly in line with the condenser. This location was the only one available on the canal and ideal in that it allowed the shortest run from the canal to the condenser. However, it did involve construction problems due to interferences with existing facilities that had to be maintained in service.

A portion of the existing ash hopper building adjoining the original boiler room was removed with the initial demolition work, but it was necessary to maintain either the railroad track or the roadway leading to the fly ash hoppers in continuous service. The travel of the existing coal barge unloader was shortened to clear the new intake, and it was also necessary to shore up an existing overhead coal conveyor bent and the continuous unloader conveyor, as well as several conduit banks, during construction.

The intake structure (Fig. 4) consists of four components—a headworks on the canal, four tunnels as a single box section, a screen well and a pump and lube oil storage room overhead of the tunnels. The entire structure is of reinforced concrete construction and doweled into the east basement wall of the station building, thereby providing lateral support for this wall. Because of this rigid connection to the building which is supported on caissons to bedrock, the intake was supported on wood piles to prevent any differential settlement. Piles used were part of the wood piles that had formerly supported the old ash hopper building and other structures. They were within the intake area and could be re-used to support the new structure. New piling was driven into the hardpan underlying the structure at approximately elevation -33 feet to obtain 25-ton bearing capacity.

The headworks is in line with the dock at the canal, and the existing timber fender system was continued as protection against the impact of incoming coal barges. The existing sheetpile bulkhead was keyed into the new structure and serves as a bottom cut-off wall. An inclined traveling trash rake is provided to keep the bar grizzly clear of debris which has been a problem to Commonwealth on its river intakes. Because of space limitations, the rake is designed to straddle the existing coal barge unloading conveyor, and it is equipped with a steel hopper to contain the debris for periodic manual unloading.

A separate traveling screen is provided for each of the four intake tunnels. The screens have 8 foot basket widths and are equipped with spray nozzles on both sides—again because of the bad debris problem. Screen refuse troughs feed into an 18-inch pipe connecting to the discharge system, but provision has been made for the possible future installation of a grinder. The screen house arrangement is rather tight because of road and railroad clearances on either side. Access is provided by rolling steel doors the full length of the front wall. Two removable roof hatches permit pulling of the screens by a railroad crane on the adjacent track.

Quotations were received and layouts made on both wet and dry well cooling water pumps, for both two- and four-pump arrangements. The two-pump drywell scheme proved to be the most economical. Pumps selected are the vertical, single-stage, volute, mix-flow type, each having a rated flow of 106,000 gpm for a 25 foot total dynamic head. They are mounted in a pit

below the basement floor level. Removable floor framing and an open hatch in the two floors overhead permit servicing by the turbine bay bridge crane.

Center walls of each pair of the four intake tunnels are discontinued to form two separate suction wells, and these are connected to the pumps through 72-inch bell-mouthed steel thimbles cast in the basement wall. The pump discharges are connected to the condenser inlet flanges by steel transition piping embedded in the building foundation mat. Butterfly valves are provided on both the suction and discharge sides of the pumps to permit taking one pump out of service while continuing operation with the other.

The pump and lube-oil storage room location over the top of the intake tunnels was the solution to several layout problems. First, there was not space available in either the station building or the screen house for the service water and screen wash pumps. Second, the Building Code required a fireproof vault for the lube-oil storage tanks. By closing in the area between the screen well and the east basement wall with two side-walls and a roof slab, ample space was provided for both the pumps and tanks. By adding a concrete partition wall and fire-doors, a completely fireproof vault was obtained for the oil tanks as well. This location for the pumps provided positive suction from the intake tunnels through spools embedded in the floor slab, and since the flow demand on these pumps is a small percentage of the total cooling water flow, there is no significant effect on the system.

Construction of the intake structure was carried out in two stages (Fig. 5) because of the need to keep the dock area open for fly ash removal. The first stage included the west half of the structure from the screen well to the

FIG. 5 - CONSTRUCTION OF COOLING WATER INTAKE STRUCTURE.

building basement wall; the second included the remainder of the structure, out to the canal. Individual sheetpile cofferdams were driven for each stage. The sheetpiling was driven to seal into the hardpan and cross braced across the excavation using steel struts supported from soldier piles. The cofferdams proved to be very tight and a single 3-inch pump, working only intermittently, was sufficient to take care of seepage. The new piles inside the cofferdams were driven using a set of loose leads supported by a crawler-mounted rig located at grade.

Discharge Shaft and Tunnel

As a result of the economic study previously described, the new plant cooling water discharge was combined with the station's existing north system by construction of a new vertical shaft and a connecting tunnel.

As shown in Fig. 4, the shaft has an inside diameter of 9 feet and is about 80 feet deep. It is centered on the twin outlet flanges of the condenser and connected to them by a steel "wye" transition piece.

The top half of the shaft is in earth, predominantly clay, and the remainder is in the limestone bedrock. For the earth portion, minimum concrete wall thickness was specified at 2 feet. This dimension established the pay lines for the excavation and concrete work.

The top section, in the earth, is lined with 3/8-inch steel plate anchored to the concrete wall by bar straps and butt-welded to the steel transition piece at the top of the shaft.

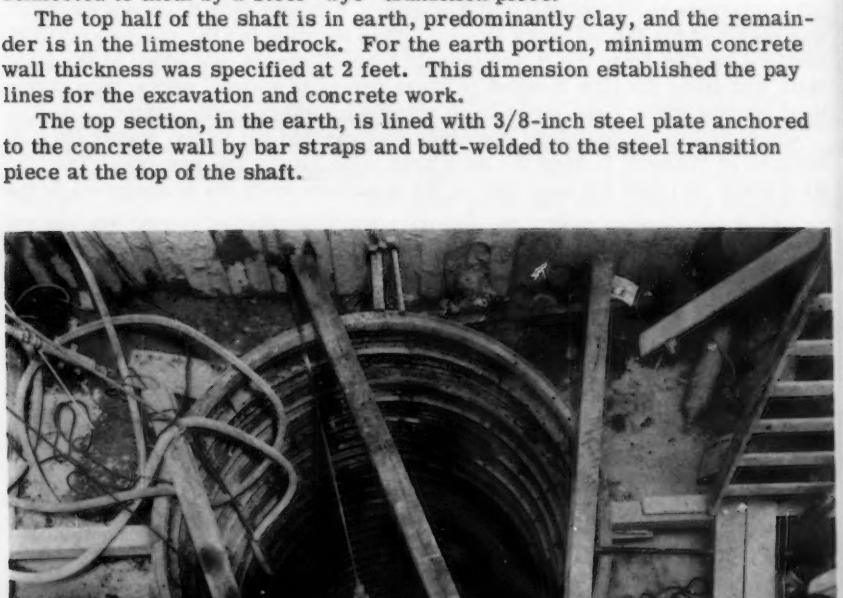


FIG. 6 - COFFERDAM FOR COOLING WATER DISCHARGE SHAFT.

The lower, rock portion is unlined, and its concrete wall thickness was specified at 9 inches minimum and 12 inches average, with the latter figure defining the pay lines. The vertical shaft ends in a "tee" transition at the new rock tunnel which connects the shaft to the existing discharge tunnel. The new tunnel is 270 feet long and has the same 9 foot inside diameter and concrete lining thicknesses as specified for the shaft.

Construction of the shaft and tunnel was complicated by old boilerhouse piling and masonry foundations and also by the fact that blasting had to be controlled to prevent possible damage to the Unit 18 boiler and turbine which were immediately over the new tunnel.

Existing masonry foundations were broken out, and the shaft was excavated using a corrugated steel cofferdam (Fig. 6) braced with integral circular steel walers. Old piles were not pulled for fear of disturbing the ground. They were cut and removed instead as excavation progressed. Roch was hit at -50 elevation; then blasting began.

All blasting was checked by a portable seismograph mounted on the Unit 18 turbine. The seismograph was started recording in advance of each shot so that both normal turbine vibration and the increased vibration due to blasting were plotted on the same film. Films could be developed and interpreted within four to five hours, if necessary, so that if excessive vibrations were indicated, the next round could be reduced.

No additional vibrations over the normal turbine vibrations were recorded in blasting the rock section of the shaft. The shaft rounds pulled an average of 3 to 4 feet using from 25 to 30 holes. Millisecond delays were used—6 to 8 delays with a maximum charge of 16 pounds of powder per delay.

The tunnel was driven by the top heading and bench method with the heading advancing 15 feet ahead of the bench and pulling one-half to two-thirds of the tunnel face. For the first 60 feet of tunnel, 6 foot holes were drilled in an effort to pull more muck and speed the work. However, the holes did not break satisfactorily and slight additional vibrations were experienced on the turbine. Drill hole depth was therefore cut back to 3 to 4 feet, and 6 to 8 delays were used with a maximum charge of 10 pounds of powder per delay. Better progress resulted, and no appreciable increase in vibrations was recorded except for the last 50 feet of tunnel, directly under the turbine. In this last 50 feet, the seismograph measured a maximum total displacement of 3.2 mils compared to the 0.6 mil average displacement it had been indicating for the normal operation of the turbine. The seismograph proved to be an effective and economical means of checking blasting operations.

Concrete walls and liner plate for the top of the shaft were placed after the first four rounds in the rock section of the shaft had been shot. Concrete lining for the remainder of the shaft and the tunnel was pumped in place (Fig. 7) after all rock excavation had been completed and trimmed. Concrete was specified to have 3000 psi strength, 3/4-inch maximum aggregate and 4 to 6 inch slump. Bleeder pipes were grouted into any rock fissures which were running water and capped prior to placing concrete to prevent washing out the lining. All rock broke well in the tunnel; timbering was not required.

Total quantities for the discharge shaft and tunnel included 340 cubic yards of earth excavation, 1300 cubic yards of rock excavation and 560 cubic yards of concrete.

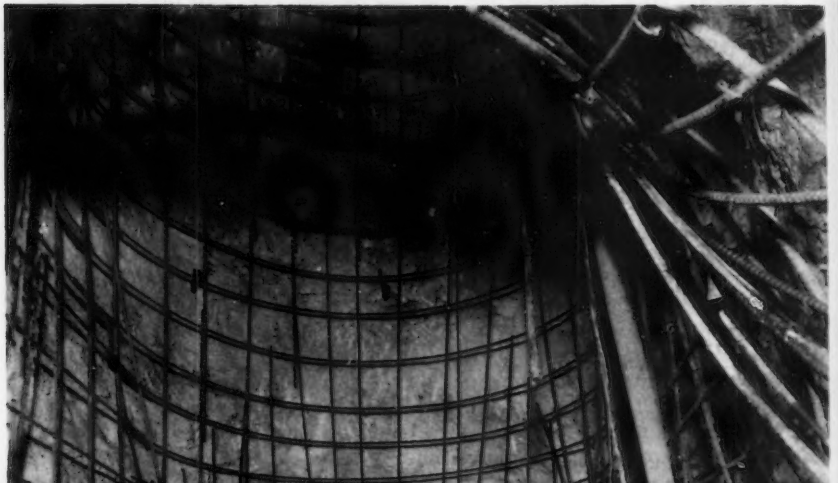


FIG. 7 - COOLING WATER DISCHARGE TUNNEL AT SHAFT TRANSITION.

Foundations

To augment existing soils data on the site, ten new borings were made in the new Unit 19 area. Logs of these borings, supplemented by laboratory tests on representative undisturbed core samples, indicated presence of loose fill material at the ground surface, underlain from elevation -13 to -23 feet by loose silt and compressible clays with some intermediate thin sand layers. Underlying these compressible materials are tough to very tough clays, followed by hardpan at elevations -28 to -33 feet and bedrock (dolomitic limestone) at an average elevation of -50 feet. Groundwater was

encountered near elevation -1 foot and was assumed at 1 foot above the river level.

As is common in the entire Chicago area, all foundations were carried either to hardpan or bedrock to avoid settlements due to the loose and compressible materials in the upper strata. The Chicago Building Code permits maximum bearing pressures of 12 kips/sq ft and 200 kips/sq ft respectively on the hardpan and bedrock.

Station Building Foundation

The station building foundation is a continuous reinforced concrete mat over the entire plant area, supported on concrete caissons to bedrock. Caissons are located under each building column, under the turbine pedestal piers and under each of the coal pulverizers.

The mat is stepped at two different elevations (see Fig. 3) to suit the lower basement level in the turbine and auxiliary bays and the higher grade level in the coal-handling and boiler bays. A continuous, pilastered wall connects the two mat levels and carries the column loads at this line down to the supporting caissons. Pits are provided in the mat for the cooling water pumps, ash hoppers and other equipment. The bottom of the mat is depressed locally at these pits to provide full continuity.

Building column base plates are recessed and anchored into the top of the mat except for the line of columns adjacent to the existing Unit 18 boiler. These columns were supported directly on their caissons since they had to be erected in advance of the mat construction to support an existing masonry wall so that the demolition of the old boiler house could be completed.

The foundation mat is designed essentially to span as a one-way slab in the north-south direction between parallel lines of caissons running east and west. Closely spaced bands of reinforcing are located in the mat over the caissons to provide continuous local beams to carry the slab loading between the individual caissons. Mat thickness is 4' -6" in the turbine and auxiliary bays; 4' -0" in the coal handling bay; and, in the boiler bay, both 3' -0" and 4' -0" with depressed areas adjacent to the deep ash hopper pits being 8' -6" thick. The top of the mat is finished with a nominal 6-inch topping sloping to the floor drains. This topping served to embed trench seat angles and other finish inserts. All equipment pads are doweled into the mat, and some conduit banks and process piping were also cast into the mat pours. The low basement area in the turbine bay is protected against flood water up to elevation +8 feet; continuous steel water stops are provided at all construction joints.

Ninety-six caissons were used to support the station building foundation, and four additional caissons were used under the main transformer and the two new crane columns in the old turbine bay. Caissons range from a minimum of 4' -0" to 9' -6" in diameter and were sized for the total loads supported on the basis of an allowable compressive stress of 750 psi on a cross-sectional area of 3750 psi concrete. Allowable compressive stress for the caissons supporting the turbine pedestal piers was reduced to 450 psi, in keeping with the more conservative design criteria established for this structure. The larger caissons are reinforced for their full length, while the smaller ones are reinforced in their top halves only; all have matching dowels projecting into the mat.

Wells for the caissons were hand excavated by the conventional "Chicago

method," using timber lagging braced with steel rings. Bottoms of the wells were cut into the bedrock to obtain level bearing; 25 percent of the caissons were required to be drilled into the rock a distance of 10 feet.

Yard Foundations

All foundations for the yard equipment, including stack, I.D. and F.D. fans, precipitators and auxiliary transformers are supported on cast-in-place concrete piles. Use of caissons, with their 4' -0" minimum diameter limitation, was economically impractical for these foundations because of their spread and relatively light loadings.

The pile shells were mandrel-driven to penetrate 2 to 5 feet into the hardpan to develop a 40-ton bearing capacity. Spiral-corrugated, 16-gage, 12 inch diameter cylindrical shells were used and 3750 psi concrete was specified for the piles.

The stack foundation is an octagonal mat, 70 feet wide, having a center depth of 9' -6" which tapers to 4' -6" at its edges. The mat is supported on 340 piles and is reinforced with four-way bands of reinforcing top and bottom. Dowels are provided to match the concrete stack shell reinforcing. Several of the duct support columns are supported from the stack foundation.

Because of their size and proximity to each other, the four I.D. fans are founded on a continuous concrete mat footing, 150 feet long, 14 feet wide and 3 feet deep. This continuous mat permitted a more economical layout of the supporting piling; it also provided a greater concrete mass to resist vibrations in the event of any unbalance in the fans. Most of the duct support columns are also carried by this mat. The F.D. fans are similarly supported on a continuous mat footing—of lesser dimensions because of their closer spacing.

The precipitator is supported on individual pile footings with the exterior concrete sill walls of the precipitator enclosure spanning as grade beams between the pile caps. The foundations for the auxiliary transformers are spread mats over the piles.

Turbine-Generator Pedestal

The close-coupled, cross-compound arrangement of the Unit 19, reheat turbine-generator reflects the latest advance in turbine design to meet the demand for machines of higher capabilities and increased operating economy.

A cross-compound design physically separates the high- and low-pressure turbine sections with each section driving its own generator. This separation permits a lower speed to be used for the low-pressure section. The lower speed, in turn, allows the use of large, last-stage buckets and a liberal exhaust area—the two features most vital to the efficient performance of a large turbine operating at low exhaust pressures.

By a parallel close-coupling of the two turbine sections, added efficiency is gained since the lengths of cross-over piping are reduced. More importantly, the close-coupled arrangement results in a very compact installation with consequent savings in floor space and foundation costs since the two turbine sections can be mounted on a common foundation pedestal.

The design of this single pedestal to support, in effect, two separate turbine-generators having different speeds presented an unusual and challenging structural problem.

Pedestal deck inside dimensions were dictated by the bearing area and clearance dimensions of the machine. The layout below the deck was governed by physical clearances on the condenser, electrical bus runs, heaters and the major steam and allied piping. The condenser is installed with its longitudinal axis directly in line with the shaft of the low-pressure unit. It is spring-supported from the foundation mat and rigidly bolted to the low-pressure turbine exhaust flange. Two low-pressure feedwater heaters are mounted in the condenser neck perpendicular to the condenser axis.

An outline of the turbine pedestal is shown on Fig's. 8 and 9. It has three longitudinal continuous piers which are broken only by access ports at the basement level and by openings required for the steam piping and heater tube-bundle removal. In the transverse direction, the crossbents differ for each turbine unit and none, except the one on the east face, are continuous across the full pedestal width.

Machine loads, as given by the turbine manufacturer, are tabulated on Fig. 8. Vertical static loads include a condenser operating load of 750 kips, and total 3540 kips. Lateral loads are reactions to frictional forces resisting the thermal expansion and contraction of the machine parts. At the two generator stators and the low-pressure turbine, lateral loads equal the normal static loads which is equivalent to assuming a friction coefficient of unity. The manufacturer stated that these thermal loads were empirical and allow for the added normal forces produced by pre-tensioning the anchor bolts as well as unpredictable differential expansion conditions.

The turbine manufacturer also specified that to maintain machine alignment, maximum vertical or horizontal displacement of machine bearing surfaces (for the most critical combination of loads) should be limited to 20 mils (0.020 inches). This value is the sum of all deflections in the supporting members due to bending and shear plus pier shortening. In addition, the manufacturer recommended vertical and lateral impact factors and allowable working stresses.

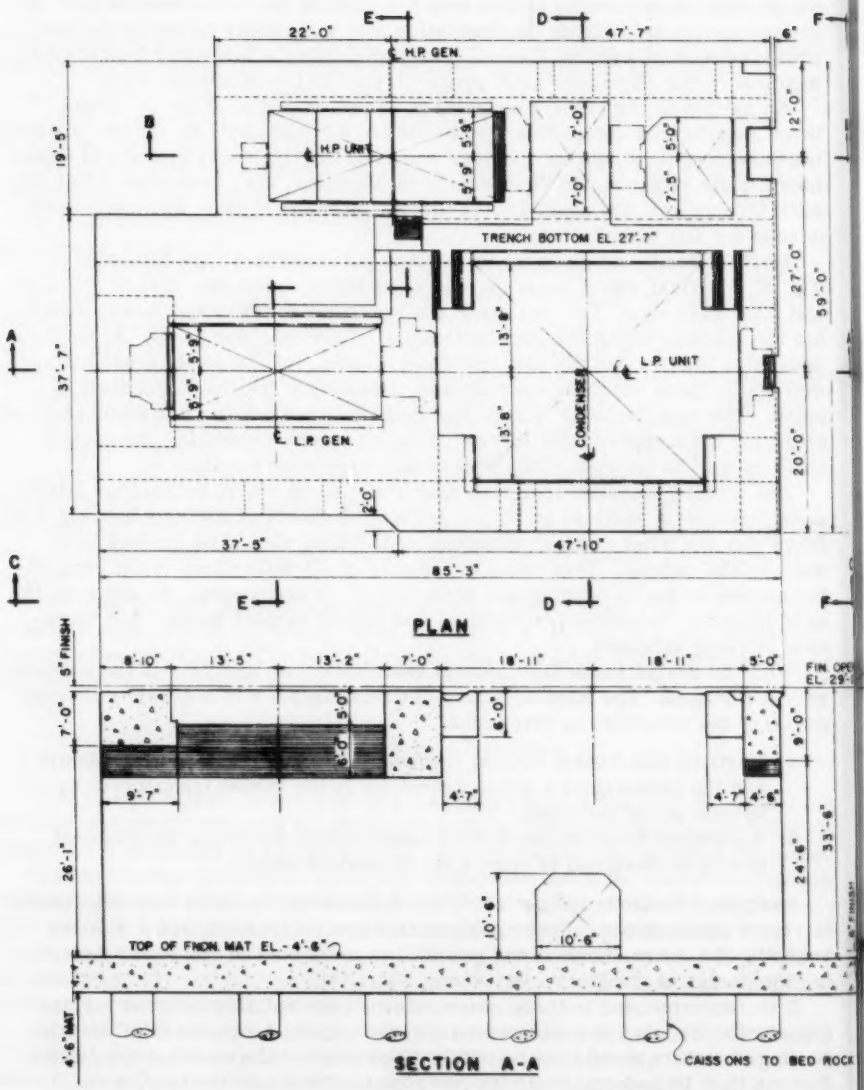
With the design loads and criteria established, an analysis of the pedestal was undertaken. The first objective of this analysis was a dynamic investigation of the structure to determine:

1. Vertical and lateral natural frequencies of individual pedestal bents and the pedestal as a whole compared to the forced frequencies or speeds of the machine.
2. Equivalent vertical and lateral static forces for which the pedestal should be designed to assure its dynamic stability.

Analytical methods for such a dynamic investigation have been suggested in recent publications. These publications are referenced, and a similar analysis of a more conventional pedestal is presented in the Paper describing the design of the Morro Bay Steam Plant.(1)

It is recommended in these same publications and also by most turbine manufacturers, that to avoid resonance, the natural frequencies of the supporting structure should not be within 20 percent of the machine frequency. For the Unit 19 pedestal, with its two speeds—1800 rpm for the low-pressure unit and 3600 rpm for the high-pressure unit—this criterion was doubly binding.

Preliminary investigation had indicated that physical clearances prohibited using members large enough to produce natural frequencies above 3600


22'-0"
E
D
47'-7"
6"
F
℄ H.P. GEN.
B
H.P. UNIT
19'-5"
5'-9"
5'-9"
7'-0"
7'-0"
7'-5"
5'-0"
12'-0"
TRENCH BOTTOM EL. 27'-7"
27'-0"
59'-0"
A
37'-7"
5'-9"
5'-9"
℄ L.P. GEN.
13'-8"
13'-8"
L.P. UNIT
CONDENSER
20'-0"
2'-0"
37'-5"
47'-10"
C
85'-3"
E
D
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PLAN
5" FINISH
8'-10"
13'-5"
13'-2"
7'-0"
18'-11"
18'-11"
5'-0"
FIN. OPE
EL. 29'-
7'-0"
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9'-7"
4'-7"
4'-7"
4'-6"
26'-1"
24'-6"
33'-8"
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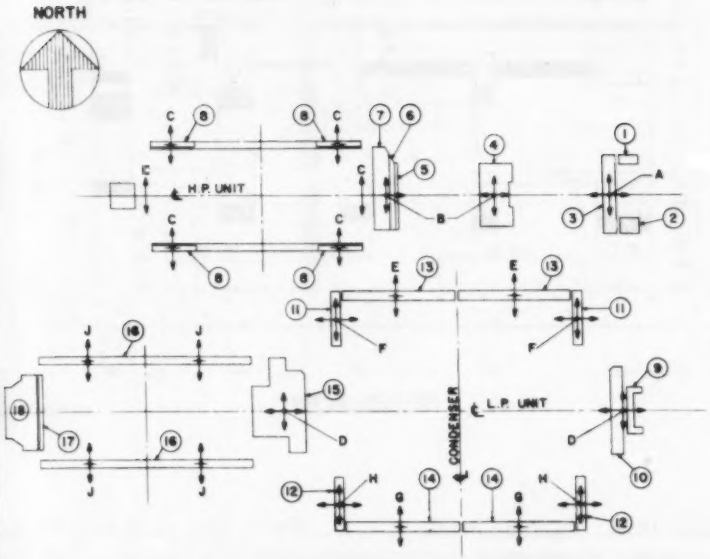
10'-6"
TOP OF FNDN. MAT EL. - 4'-6"
4'-6" MAT
SECTION A-A
CAISSONS TO BED ROCK



MACHINE LOADING PLAN

LOADING TABULATION - KIPS					
VERTICAL LOADS				LATERAL LOADS	
①	20	⑩	105	A	60
②	20	⑪	208	B	30
③	75	⑫	205	C	146
④	207	⑬	200	D	40
⑤	31	⑭	194	E	200
⑥	31	⑮	255	F	208
⑦	24	⑯	202	G	194
⑧	146	⑰	30	H	205
⑨	35	⑱	104	J	100

FIGURE 8

SHEET 1 OF 2

TURBO-GENERATOR PEDESTAL

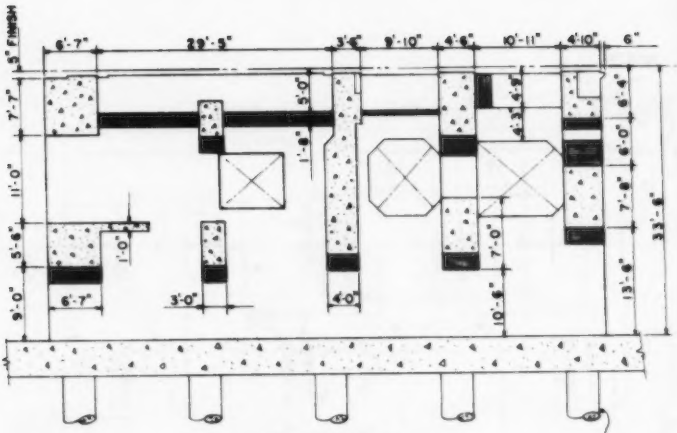
FISK STATION UNIT 19

COMMONWEALTH EDISON COMPANY

10 20

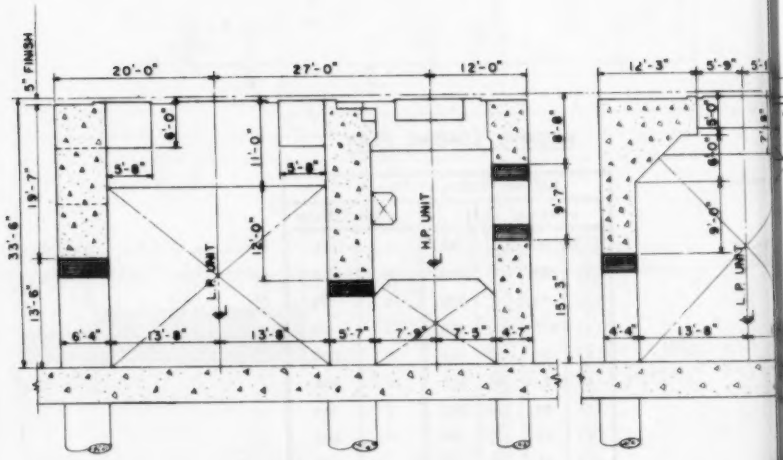
SCALE

PO 1

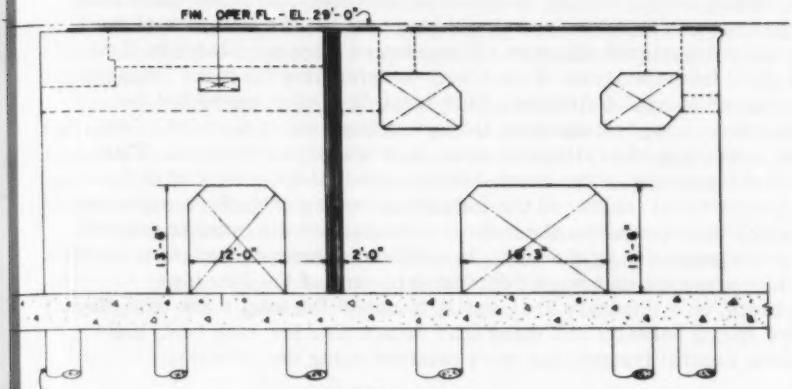


SECTION B-B

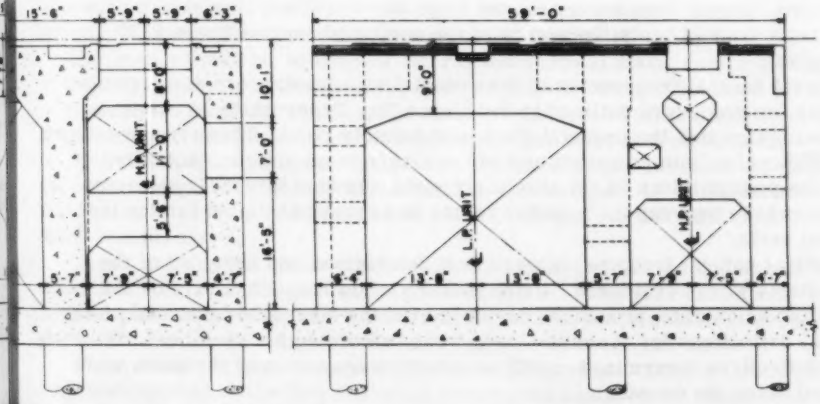
CAISSONS TO ROCK



SECTION D-D



SECTION C-C



SECTION E-E

SECTION F-F

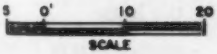
FIGURE 9

SHEET 2 of 2

TURBO-GENERATOR PEDESTAL

FISK STATION UNIT 19

COMMONWEALTH EDISON COMPANY



cycles/min. Low tuning was therefore used and members were reproportioned where required.

In calculating lateral natural frequencies, only the four cross-bents under the high-pressure unit and the one continuous bent (on the east face of the pedestal) were considered effective. Frequencies were calculated by first making a rigid frame analysis of each bent to determine the force required to produce a unit lateral deflection. This force "k," when corrected for shear deformation, is, by definition, the spring constant of the bent. The total mass load engaged in vibration of the bent was then computed. This load included the weight of the cross-beams, one-half the weight of the columns (proportional lengths of the continuous longitudinal piers were considered acting as columns for the individual bents) and the tributary static machine loads supported by the bent. In addition, corresponding mass loads from the low-pressure unit were distributed to each of the four high-pressure bents, on a tributary area basis, to obtain the total mass load "W."

With the spring constant and mass load determined for each bent, individual lateral natural frequencies were obtained using the formula:

$$n_e = \frac{1}{2\pi}\sqrt{\frac{k}{W/g}}$$

where g = acceleration due to gravity.

Lateral natural frequencies ranged from 630 to 1310 cycles/min for the four single framed bents; frequency of the combined east bent was 1320 cycles/min.

Lateral natural frequencies of the pedestal as a whole were also calculated using the procedure outlined in the Morro Bay Paper which is based on the assumption that the pedestal deck is absolutely rigid. These frequencies were 770 cycles/min minimum and 945 cycles/min maximum. Actually, since the pedestal deck is not absolutely rigid, the true lateral frequencies lie somewhere between these latter values and those calculated for the individual bents.

Vertical natural frequencies were next determined and here, all of the cross-bents were considered. Using either a rigid frame analysis or the simplified calculation procedure developed for the Morro Bay pedestal, total vertical deflections for the mass loads were calculated for each bent. With these deflections determined, vertical natural frequencies of the bents were obtained using the formula:

$$n_e = \frac{187.7}{\sqrt{\Delta}}$$

For the three bents supporting the low-pressure unit, vertical natural frequencies were 1400, 1490 and 1570 cycles/min with the last value (for the bent supporting the exciter) being marginal. Frequencies for the high-pressure unit bents were all below 2640 cycles/min.

These calculated lateral and vertical natural frequencies are, of course, approximations only due to the highly indeterminate nature of the problem and the simplifying assumptions made to solve it. In this regard, it should be noted that all frame piers were assumed as fixed-ended and the modulus of elasticity of the concrete was assumed to be 620×10^3 kips/ft^2 (n = 7) in

all frequency calculations. These assumptions are both on the safe side when low tuning is being used.

It was concluded that the probable range of natural frequencies of the pedestal, in both directions, was sufficiently less than the two forced machine frequencies to preclude any resonant response to possible machine vibrations.

The next step in the analysis was to determine the equivalent static vertical and lateral forces for which the pedestal should be designed to assure its dynamic stability. The turbine manufacturer had recommended that 125 percent and 25 percent of the machine loading be used as equivalent static vertical and lateral design forces respectively.

Of the publications referenced in the Morro Bay Paper, one[2] gives the magnitude of these equivalent static design forces (P_s) as functions of the weight of the machine rotating parts and the ratio of the natural to forced frequencies of the pedestal, by the formula:

$$P_s = 4\,L \left[\frac{\left(\frac{n_e}{n_m}\right)^2}{\left(\frac{n_e}{n_m}\right)^2 - 1} \right]$$

Where L = weight of machine rotating parts and n_e and n_m = natural frequency of pedestal and machine frequency respectively.

Using this formula, equivalent lateral and vertical static design forces were both determined as the sum of the individual forces calculated for each of the units. These forces checked closely with those recommended by the manufacturer and his recommended percentages were therefore used.

The first objective of the analysis—a dynamic investigation of the pedestal—was completed. The second objective was an analysis of the structure for the most critical combination of loads to determine the maximum deflections and stresses.

First, each of the seven cross-bents was analyzed for the equivalent vertical static forces (125 percent of the machine loading) plus dead load. Maximum moments, shears, axial loads and deflections were obtained; vertical deflections were all considerably less than the allowable 20 mils.

Then, the equivalent lateral static force (25 percent of the machine loading) was applied to the five resisting transverse bents on the basis of either a rigid or flexible deck. For the rigid deck assumption, the force was distributed to each of the bents in the ratio to their spring constants (as calculated for lateral natural frequencies). For the flexible deck, the force was distributed to the bents on the basis of tributary area loadings. The individual frames were analyzed for the maximum lateral force of the two cases, and maximum moments and shears obtained.

Lower cross-beams and piers of the bents were designed for the combined moments, shears and axial loads due to the vertical plus lateral equivalent static forces.

Deck members, however, were designed for the combined moments, shears and axial loads due to the equivalent vertical static forces plus the thermal lateral forces (thermal forces, at 100 percent of the static loading, being greater than the 25 percent equivalent lateral static force). Members were

actually designed on a trial and error basis using transformed areas. Unit concrete and reinforcing steel stresses were calculated first for the thermal forces acting horizontally; the added stresses were then calculated for the vertical forces.

Combined stresses for all deck members, as finally proportioned and reinforced, were below the allowable. Deflections were also computed for the combined loading, and all were less than the allowable 20 mils.

Total quantities for the pedestal, as constructed, included 1970 cubic yards of concrete, 280 tons of reinforcing steel and 26 tons of miscellaneous iron.

Because of the large volume of concrete involved, shrinkage was of considerable concern. To minimize shrinkage, a minimum amount of Type II cement and 2 1/2-inch maximum aggregate were specified for the 3000 psi concrete mix. Also, maximum concrete temperature during placing was limited to 60 degrees. Meeting this latter specification required replacing about half of the mixing water with ice.

Another notable feature was the liberal use of number 14 and 18 reinforcing bars. These bars provided the required steel areas with minimum placing costs and also gave added clearances for the large size aggregate. Piers were poured individually to construction joints at the soffits of the deck cross-beams; the complete deck (Fig. 10) was poured in a single pour.

FIG. 10 - TURBINE-GENERATOR PEDESTAL - OLD TURBINE ROOM IN BACKGROUND.

Superstructure

As shown in Fig. 3, the four functional bays of the station building are of different heights but are framed as an integral structure to resist vertical and lateral loadings.

Dismantling clearances for the turbine and other equipment dictated the use of a 100-ton bridge crane having a clear span of 93 feet. As noted previously, one of the advantages of the new plant layout was that its crane runway could be extended out into the old turbine room to utilize this area as laydown space for both the new and existing units. Design of this crane runway extension involved several rather unusual framing problems.

First, a clear span across the 65-foot width of the old turbine room was desired, and it was accomplished by deep plate girders stayed laterally by horizontal trusses. Second, the existing turbine room roof trusses were supported by five columns (see Fig. 10) which would block passage of the new crane; the columns had to be removed and an alternate support system provided for the existing trusses. This problem was solved by picking up each of the old trusses with a pair of channel outriggers, connected to the vertical web members of the two new adjacent turbine bay roof trusses. Prior to transferring the 70-kip end reactions of the old trusses, the columns were jacked up to compensate for the added deflection of the new trusses and then burned off after the outriggers had been connected.

The turbine bay column spacing was carried into the other three bays for framing symmetry and to minimize interferences with piping and conduit runs.

Vertical wind posts are slip-connected to the end turbine bay roof truss to resist the wind loading on the east wall, and this loading is carried down to the longitudinally braced column lines by a system of sway frames and top chord bracing in the roof trusses. All transverse lateral loading in the turbine and auxiliary bays is transmitted to the north-south braced bents in the boiler bay by diaphragm action of the concrete floor slabs at the operating and feeder floor levels.

The deaerator is supported from a framing grid of built-up plate girders and rolled 36 WF sections. Lubrite plate assemblies are used under one of its two saddles to allow for the thermal expansion of the vessel. An open tiered system of framing is located overhead of the deaerator and coal bunker as a skeleton framework to support hangers for the main steam and reheat lines and the extended leads from the boiler to the main steam and hot reheat risers.

The boiler is hung from a steel framing system at elevation 190' -4" to accommodate its 8-inch, downward, thermal expansion during operation. Only the hanger and buckstay steel was furnished with the boiler; all supporting steel and platform framing was included under the building structural steel contract.

As shown in Fig. 3, the single, 70-foot long boiler drum and the gas pass cross-over sections of each of the twin furnaces did not permit running the interior boiler bay columns up to the top support steel level. This required the use of 15-foot deep plate girders for the two 65-foot spans between the three columns in the center of the boiler bay. The total reaction of the top support steel system to the middle column is about 4000 kips. This load, plus the load from the boiler bay roof (which is posted up from the top support steel), plus the added loadings of the boiler platforms and air preheaters

below, add to a total column loading of 5200 kips. The column is a 14 WF 320 core section with two 30" by 4-1/4" cover plates.

To provide added lateral stability to the structural framing supporting the boiler "pendulum," the vertical bracing in the boiler bay was designed for tributary lateral forces equal to 2 percent of the total hanger loads (applied at the top steel level) in addition to normal wind loading. A system of lateral guides was integrated with the platform framing to maintain the boiler in vertical alignment when subjected to lateral thermal expansion.

Below the boiler drum and cross-over section levels, four intermediate columns were provided at each furnace to reduce the spans, since the depths of the platform framing members were limited by the low headroom between platform elevations. The layout and framing design for these platforms was further complicated by the boiler's 190 soot-blowers, about half of which are the retractable type.

A total of 4900 tons of structural steel was erected. Shop connections were riveted; all field connections were made with high strength bolts.

Coal and Ash Handling Systems

Fisk Station is a coal burning plant and its coal handling facilities are quite extensive. Coal is brought into the plant either by the overhead conveyor from the Company's Loomis Street coal yard (west of the station, see Fig. 1) or by the overhead conveyor on the dock along Mason's Canal. This latter conveyor can be supplied directly from railroad cars unloaded by a rotary car dumper or from barges in the canal by a traveling barge unloader and its feeder conveyor. Conveyors from both systems terminate at transfer towers where coal is distributed to a series of other conveyors feeding bunkers for the various units.

Existing coal handling facilities were quite adequate for Unit 19. They required only the addition of a new conveyor system, fed from the ends of the two existing conveyors serving the Units 17 and 18 bunkers.

In preference to other arrangements studied, a single conveyor, 130 feet long, was selected. It spans overhead of the turbine and auxiliary bays to the west end of the new bunker. It is inclined at an angle of 17 degrees and is supported by two steel trusses seated on rocker frames posted from the turbine bay roof at their lower ends. At the head end, the trusses are supported from framing cantilevered from the bunker face.

Coal is fed from this conveyor through a transfer chute to a motor-operated tripper conveyor running the full length of the new coal bunker. The tripper has double spouts discharging through continuous, self-closing, rubber-sealed slots in the bunker top slab. Both conveyors have 48-inch belts and a capacity of 1200 tons of coal per hour.

The coal bunker is 35 feet wide by 130 feet long with a storage capacity of 5000 tons. It is compartmented to form eight separate bottom hoppers—each feeding an individual mill. Based on Commonwealth's past experience with the coal to be used, hoppers were designed for 60 degree minimum side slopes and lined with 3/16-inch thick stainless steel wear plates on their lower portions. Bunker framing is of all-welded construction with integral wide-flange and channel stiffeners. The vertical bunker plates are framed to the inside flanges of the supporting columns, and, together with the continuous top and bottom horizontal beams taking the stiffener reactions, they span as

plate girders between the columns to carry the bunker vertical loading. The entire interior of the bunker above the hopper wear plates is lined with 2-1/2 inches of gunite for corrosion protection.

The coal bunker has a carbon dioxide deluge fire protection system with nozzles located at the top of the bunker and at the bottom hoppers. A vacuum type dust collection system is also provided to remove coal dust and any gas built up over the coal pile inside the bunker to reduce explosion hazard.

Although Unit 19 is equipped for gas operation, it is expected to be primarily a coal burning plant. At its rated boiler capacity of 2,200,000 lb/hr, fuel consumption will be 145 tons of coal per hour—some 3500 tons per day.

Ash Handling System

Commonwealth uses Central and Southern Illinois coals in its plants. These are bituminous coals having heat values of about 10,000 Btu/lb but relatively high sulfur and ash contents of 5 and 15 percent, by weight, respectively. The high sulfur content accounts for the elaborate bunker lining design previously described. The ash value is the factor which dictates need for the large number of boiler soot-blowers and also necessitates a rather extensive ash handling system.

At 15 percent of normal coal firing rate of 145 tons per hour, the boiler turns out 22 tons of ash per hour; 4 tons of which is bottom ash and the remaining 18 tons is fly ash. Water jet pumps are used to pump the bottom ash as a slurry to an adjoining property where a complete ash handling system for the entire station is located. There it is dewatered and collected in bins, then removed to dumps.

The fly ash is discharged with the flue gas through the boiler ductwork to twin electrostatic precipitators which have a recovery efficiency of 98 percent. Fly ash is retained in 16 bottom hoppers of the two precipitators, then pumped pneumatically to new storage silos, which will also serve several of the existing units.

The large volume of fly ash produced at the Fisk Station is a salable by-product which is sold commercially as a pozzolan material for concrete. Of incidental interest is the fact that all concrete used in this project was specified to contain 20 pounds of fly ash to each sack of cement, in addition to 3 to 6 percent of air entrainment.

Stack and Ductwork

The problem of air pollution by industrial waste gases is one that has received much attention in recent years. This same problem must be faced in the design of a modern power plant stack. It is all the more acute when the plant is coal-fired and located in a densely populated area as is the case with the Fisk Station.

Commonwealth Edison has been extremely conscious of air pollution and has sponsored considerable research on the problem in an effort to develop stack design criteria over and above existing code requirements.

These research programs, initiated in the early thirties, have been concerned primarily with the behavior of stack discharge plumes and, more specifically, with the possibilities of downwash of the plumes, which results in the gases being brought down to the ground prior to dispersal. This problem is a complex one with many factors involved; it can best be studied by

model wind tunnel tests. Even then, it is not a matter of precise determination but rather one of probabilities since it is impossible to duplicate all meteorlogical effects in a wind tunnel.

Wind tunnel tests have been run on models of both existing and proposed stack installations to determine the critical wind velocities above which aerodynamic downwash of the stack gases might occur for various combinations of stack height, exit velocity and gas temperature. In addition, the model test results for existing installations have been verified by field observation.

Such a testing program was conducted for a recent stack installation at Commonwealth's Crawford Station and data from this study were made available for use in the preliminary design of the new Unit 19 stack.

Design of this new stack then became a problem, basically, of selecting the optimum combination of stack height and diameter, as affecting exit velocity, which would satisfy code requirements as well as minimize plume downwash for the prevailing winds at the site. The selection also had to consider stack economy, as related to height and diameter, versus fan and fan energy costs, which are, of course, a function of the stack dimensions.

The high-efficiency precipitators used in the Unit 19 installation will limit the total particulate matter discharged from the stack to a figure well below the maximum specified by the existing Chicago Air Pollution Code. The net hourly emission rate of particulate matter will also comply with the limitations of the Proposed Comprehensive Amendment to the Chicago Zoning Ordinance.

Downwash of the trailing stack plume was next considered. Neighboring buildings and structures were studied and a trailing plume not lower than 325 feet above ground level appeared to be acceptable. The Crawford study indicated that this was a reasonable criterion and one that was attainable under virtually all normal wind conditions.

By modifying the data contained in the Crawford study to apply to the Unit 19 site, estimates of the probable plume behavior of the new stack were made for various alternates of stack heights and exit velocities. These estimated plume characteristics were plotted graphically and this graph served to define the parameter of stack sizes which would probably meet the plume criterion.

Model tests of the Fisk Station site were then conducted to verify these estimated figures and particularly to weigh the effect of the adjacent new boiler structure on the aerodynamics of the surrounding area. The model was constructed at a scale of 1:400 and covered an equivalent area of 3200 feet by 5600 feet as limited by the size of the wind tunnel. Tests were run on stacks equivalent to 400, 425, and 450-foot heights at various exit velocities and for winds from the eight compass directions.

As the final step in the stack analysis, relative total costs of the induced draft system (i.e., stack and fans) for various combinations of stack height and diameter were estimated. These estimates included shell, lining and foundation costs for the stack and capitalized energy, capacity and equipment costs for the fans.

Results of the complete stack selection study were analyzed graphically. A family of curves of various stack heights were plotted on coordinates of "stack top inside diameter" versus "comparative induced draft system total costs." Curves for the basic plume height criterion were then superimposed for the wind directions, which were shown to produce minimum plume heights by the model tests.

These curves indicated that a 450-foot stack with a 14-foot top inside diameter (corresponding to an exit velocity of 115 fps) was the most economical selection which would meet the basic plume criterion.

The stack, as constructed, has an outside diameter at the base of 35' -10" tapering uniformly to an outside diameter of 16' -4" at the top. Shell thickness varies from 23 inches at the base to 29 inches at the flue openings and then to 8 inches at the top. The brick lining is set in acid-resistant mortar, backed-up with 2 inches of glass fibre, and supported from concrete corbels poured monolithically with the shell on 45-foot centers.

Ductwork

The design of the ductwork from the I.D. fans to the stack is notable in that it utilizes the ducts as box girders, cantilevered out from rigid connections to the stack shell, to resist lateral wind loading. The twin ducts each measure 12' -9" wide by 20' -0" high at the stack flue openings. The purpose of this design was to eliminate the costly expansion joints that are conventionally used at the stack face.

This scheme required no added thickness in the duct plates and permitted use of long, unbraced columns to take the vertical gravity loading of the ductwork. The connections to the stack and all support column connections to the ducts are made at the same elevation to allow vertical thermal expansion above and below this plane. Connections to the stack for each of the ducts consist of a fixed anchor at the center of the flue opening which takes the total wind shear and flexible vertical plate connections at the duct sides to take the horizontal couple produced by the wind loading. These latter plates were cold-sprung to allow for the lateral thermal expansion of the ducts.

The four evase ductwork sections connecting to the fan discharges are spring-supported at their lower ends and supported at their top ends by pins connected across a single fold expansion joint. This arrangement permits "toggle" action of these duct sections to accommodate the eccentric lateral expansion of the two individual duct systems out from their anchor points at the stack.

Ducts are constructed of 1/4-inch plate reinforced with channel stiffeners on nominal 3-foot centers. Welded wire mesh was tack welded over the tops of the stiffeners to support the duct insulation.

Architectural Treatment

Exterior architectural treatment of the plant is simple, functional, vertically-fluted protected metal siding, broken by horizontal bands of steel sash (Fig. 11). Floor slabs are finished monolithically except at the operating floor level in the turbine bay, which has a quarry tile finish.

Ventilation is provided by fans in the east wall of the new turbine bay and the west wall of the old turbine bay, in combination with gravity vents at the turbine and boiler bay roof levels.

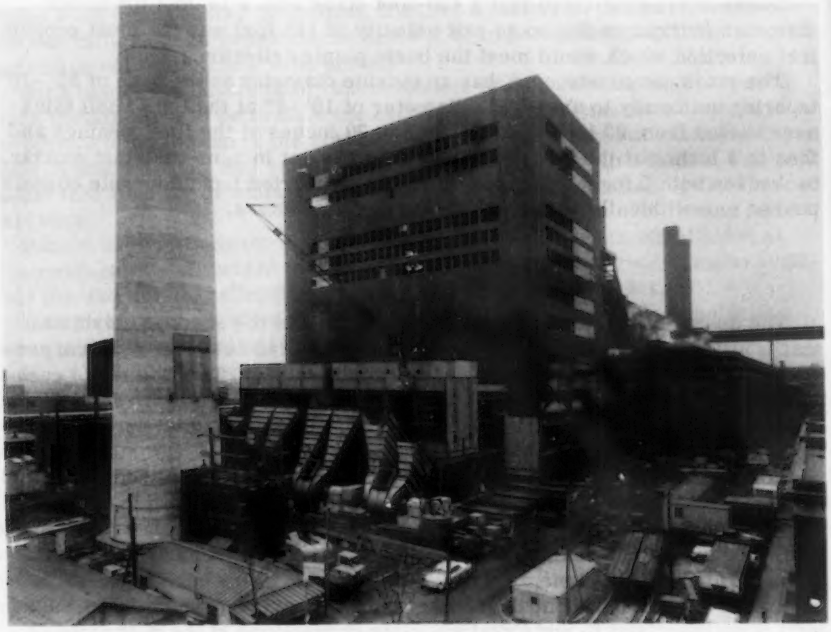


FIG. 11 - GENERAL VIEW OF PLANT DURING CONSTRUCTION.

CONCLUSION

This paper has reviewed some of the problems encountered and the solutions adopted in the design of a large single unit extension to an existing station. It is hoped that the material presented will be of some benefit to the profession, particularly to those in the steam power field in view of the current trend toward larger units in steam station design.

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Journal of the
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Proceedings of the American Society of Civil Engineers

SITE STUDIES FOR A STEAM POWER PLANT

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ABSTRACT

Irregular topography and wide variations in river flow required unusual analytical studies and comparative estimates to select the location for a steam power plant on the site, and determine elevations and arrangement of facilities. The paper emphasizes critical investigations, presents detailed estimates for each possible scheme and justifies the final arrangement.

Irregular topography and wide variations in river flow required some interesting and unusual analytical studies and comparative estimates involving selection of the location of the station on the site, elevation, and arrangement of facilities for Generating Station "D" of the Potomac Electric Power Company, now under construction. Only special investigations concerning these matters will be emphasized in this paper. Studies involving river water availability, types of circulating water systems, heat balances, thermal efficiency of equipment, life of equipment, capability charges, and capitalized operating costs were made for the purpose of determining the most economical condenser floor elevation, circulating water system arrangement, and yard layout for both initial and possible future construction. Economic analyses of this type are one of the designer's most difficult tasks, particularly since pressure is usually exerted to start working drawings and construction quickly after the plant is authorized. Some decisions must be assumed before the analyses can be completed.

This station is to be a coal burning steam power plant with an initial installation of two 1,300,000 lb of steam per hour boilers operating at a pressure of 2,486 psi gage at superheater outlet and temperature of 1,050 F with

Note: Discussion open until July 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1949 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 1, February, 1959.

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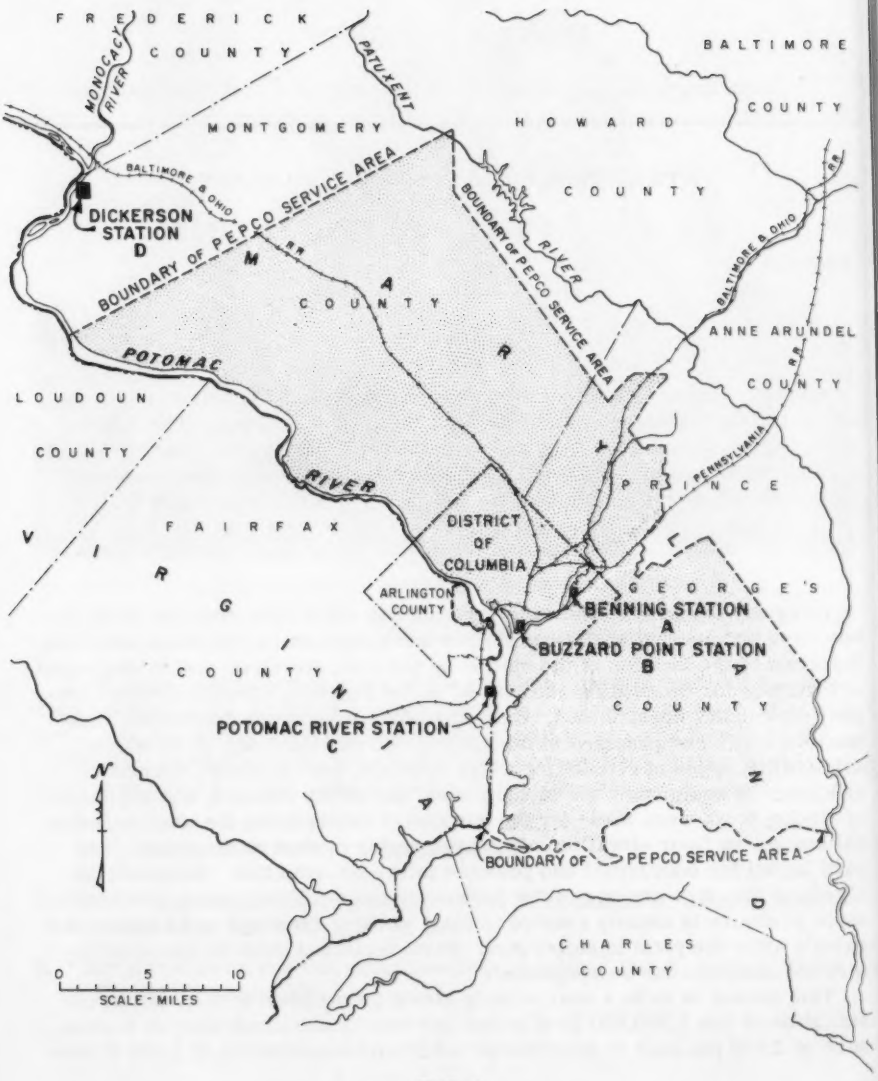


FIG. I

LOCATION MAP

GENERATING STATIONS

1,090,000 lb reheated to 1,000 F, and two 175,000 kw cross-compound single flow turbine generator units.

Site

The site was selected and purchased by the Potomac Electric Power Company in its program of planning for future power requirements. As shown in Fig. 1, it is located on the left bank of the Potomac River near Dickerson, Maryland, about 35 miles above Washington, D. C. The tract available, shown in Fig. 2, covers about 1,000 acres and the particular area selected for the station is within approximately 1 2/3 miles of the main line of the B. & O.R.R. running between Cumberland, Maryland, and Washington D.C., providing a direct route for coal from the Maryland, northern West Virginia, and Pennsylvania fields.

As this site is outside of the Company's service area and will more than supply the demands of this end of its system, future expansion of capacity may take place below Washington. It is currently thought that the ultimate

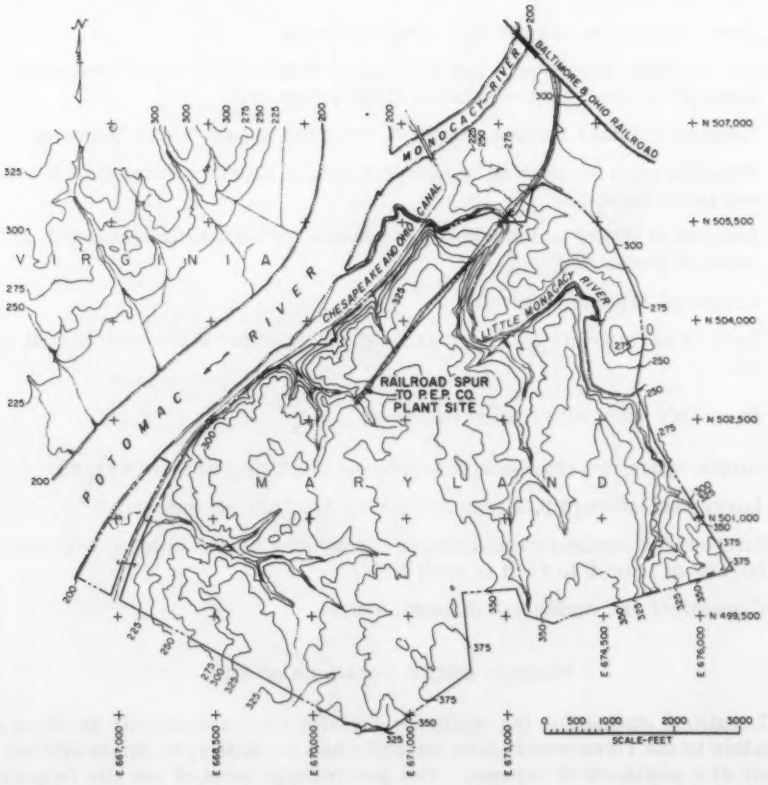


FIG. 2
GENERAL PROPERTY PLAN

PO 1

development at Dickerson may be limited to four units. Any restrictions to future expansions that have been allowed at many sites throughout the country as a result of gazing into a crystal ball have often been found regrettable; therefore, the position and orientation of the first units were established so that future possible installations to a total of six, eight, or more units would not be blocked.

All plant sites possess some features that are less desirable in the matter of cost than those at an ideal site. In this instance, they are:

Roughness of terrain, the ground surface varying generally from El. 325 to El. 215;

Large recorded variation in river level from approximately El. 190 to El. 230;

Protection required against presently predicted high water to El. 242, and possible future high water to El. 250 after construction of a proposed dam downstream;

Scarcity of circulating water from the river at times of low flow for future units;

Rock excavation, cut and fill, stepped levels;

The historic Chesapeake and Ohio canal separating a waterfront strip of about 20 acres from the balance of the property;

Lengths of intake tunnels and depth required to pass under the canal;

Possible need for cooling towers with future expansions for Unit No. 4 and later units;

Lengths of intake and discharge penstocks for cooling towers north or south of the station;

Length of railroad spur; and

Lack of navigability on the river, precluding water deliveries of coal or oil.

Favorable plant site features include:

Ample space for expansion of power plant building and switchyard;

Large coal storage pile with convenient coal car storage yard;

Interbedded sandstones and shales for bearing of foundations, with overburden of from 5 to 15 ft of soil; and

Convenient and ample ash disposal areas.

Possible Station Locations on Site

Locations studied for the station on the site were necessarily as close as possible to the river water, both laterally and vertically, to obtain cooling water at a minimum of expense. The general high level of the site falls off rather quickly near the riverbank, which is cut by gullies; therefore, selection was desirable of an area where the least leveling would be required,

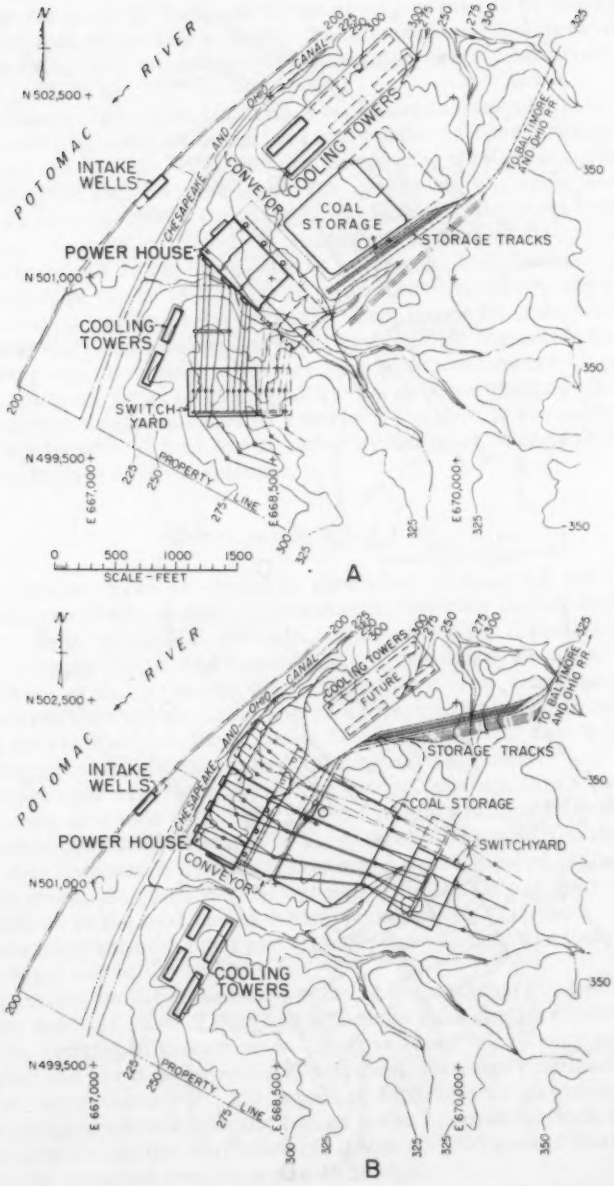
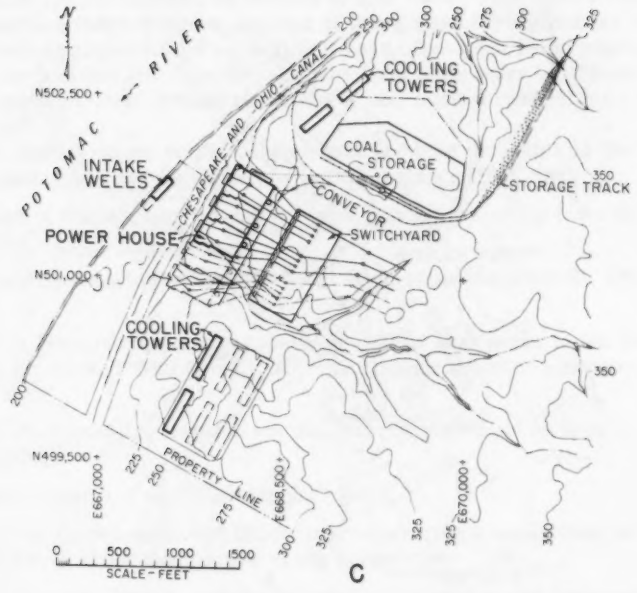


FIG. 3

ALTERNATIVE STATION LOCATIONS A AND B



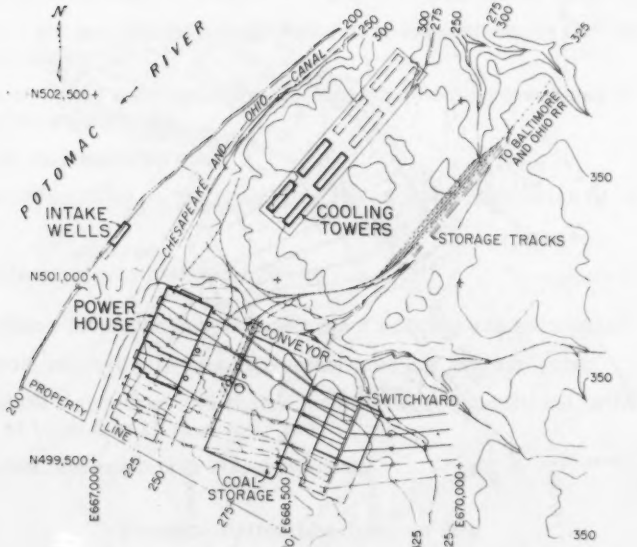


FIG. 3A

ALTERNATIVE STATION LOCATIONS C AND D

with future expansion borne in mind. Steepness of the bank elsewhere on the site indicated the general region of the southwest corner to be most practicable. This area is split by a valley. Locations facing the river north and south of this valley and along the side of the valley were considered, as shown in Fig. 3 and 3A.

A comparison of principal features of locations is shown in Table 1. Location D was discarded as being too cramped for outdoor switchyard facilities, railroad access, coal storage, circulating water and cooling tower arrangements, and future expansions. The general arrangement shown on location C was selected over locations A and B for development because of lower excavation costs, proximity of switchyard to back of station, shortness of coal conveyor and its entrance into the permanent end, ease of expansion, and relationship to intake wells. While this scheme permitted the railroad to enter the turbine room at the permanent end, it was later decided to place it at the temporary end to avoid carrying construction materials for later extensions through the initial turbine room. The disadvantageous necessity of crossing the roof with the electrical lines was outweighed by other factors. The preferred choices, indicated by asterisks in Table 1, are much more numerous for location C than for the other arrangements; none of the less satisfactory factors are inacceptable.

Development of Lot Plan

The lot plan has been developed as shown in Fig. 4 and 4A and the station cross section as shown in Fig. 5. The station has been turned slightly and moved back from the C. & O. canal to provide space for circulating water tunnels, discharge flume, and possible future penstocks, as shown in Fig. 6. The elevations of the initial and future switchyard area, trackwork, roadways, and area between the canal and river were established to utilize rock and earth excavation from the station. Rock blasting was done for Unit No. 3 to avoid future blasting close to the building and equipment.

A railroad spur crosses a 300 ft long bridge, expands into a car storage yard, and then descends on a 3 per cent maximum gradient to enter the temporary end of the turbine room by a Y. A level side spur from this track is used for cars to receive fly ash from storage space located in the lower portion of the stack. Supplies may be handled from the fly ash spur to the storage warehouse in the east end of the service building.

The sewage treatment plant and transformer oil tank are located north of the permanent end of the building.

Coal handling facilities are also north of the station. Four initial storage tracks can hold 118 cars of coal and 109 more cars can be stored in the future by the addition of three tracks. A clear track for 10 cars and a yard engine beyond the car dumper and a run-around track are provided. The coal yard is at approximately El. 323, which is sufficient so that when the conveyor is high enough to cross the tracks it can continue horizontally into the conveyor space over the coal bunkers. Over 400,000 tons of coal can be stored in the indicated area in a pile 40 ft high.

Permission was obtained to cross the presently dry C. & O. canal on temporary fills and to excavate across it for circulating water tunnels, but the canal must be cleared at the close of work. The canal may be restored to form a portion of a canal park development.

RELATIVE MERITS OF PLE

Item	Site A	
Building Excavation Units 1 and 2 Units 3, 4, 5 and 6 Units 7 and 8	$152,000) 438,000) $590,000 138,000	$98,0 34,0 30,0
	$728,000	62,0
Railroad	19,900 lin ft	8,50
Railroad Spur to Turbine Room	Enters temporary end	nters end
Switchyard	Future expansion easy. No lines cross roof. Electrical conduits to station cross ravine. Awkward angles between direction of lines and building, and between switchyard and final directio of lines.	igh t cross tall quire tance Lines roof Futur easy. tric stati
Transmission Towers Between Station and Switchyard (For 8 Units)	One bent in yard; none on building*	12 in 190 12 o
Coal Conveyors	1,700 ft	1,200
Coal Pile	Near rear of statio	etwe and
Relocation of Water Course	Required (approximately $89,000)	ot r
Cooling Towers	Good locations for towers for Units 3 and 4 south of station, balance north (North towers partially shielded.)*	Good towe out bala (Nor tial
Circulating Water Penstocks to Cooling Towers	21,700 lin ft	21,60
Intake	Expansion downstrea good*	xpan inte futu
Future Expansion	Relatively inexpensive to east*	xcav nort

*Indicates
+Later pr

TE 1
RITS OF PLE PLOT PLAN ARRANGEMENTS

e A	Site B	Site C	Site D
			Not estimated
$590,000	($98,000) (34,000) $432,000 230,000	($264,000) 163,000) $427,000 166,000	
	762,000	$593,000*	
ft	3,500 lin ft*	18,500 lin ft*	21,200 lin ft
porary	Enters temporary end	Enters permanent end*+	Enters permanent end*
pansion lines f. Elec- nduits to ross Awkward tween of lines ing, and witchyard direction	High tension lines cross coal pile. Tall towers required; long distance from station. Lines cross station roof on towers. Future expansion easy. Long electrical conduits to station.	Close to back of station; lines cross roof on towers. Large fill for future extension. Short electrical conduits to station.*	High tension lines cross coal pile; long conduit. Long electrical conduits to station.
n yard; uilding*	12 in yard (up to 190 ft high); 12 on building	None in yard; 12 on building	None in yard; 12 on building
	1,200 ft	1,100 ft*	1,100 ft
of statio	Between switchyard and station	Near permanent end*	Near rear of station; site uneven
(approxi- 89,000)	Not required*	Not required*	Bridging required
tions for or Units 3 uth of sta- lance nort owers par- hielded.)*	Good locations for towers for Units 3-6 south of station, balance north. (North towers partially shielded.)	Good locations for towers for Units 3 and 4 north of station, balance south. (North towers partially shielded.)	Distant locations
n ft	21,600 lin ft	20,800 lin ft*	28,800 lin ft
downstrea	Expansion upstream interferes with future water supply	Expansion downstream good*	Too far downstream, too near discharge
y inexpen- east*	Excavation costly to north	Relatively inexpensive to south*	Relatively inexpensive to north*

*Indicates preferred arrangement.
+Later pro[illegible]ed at temporary end.

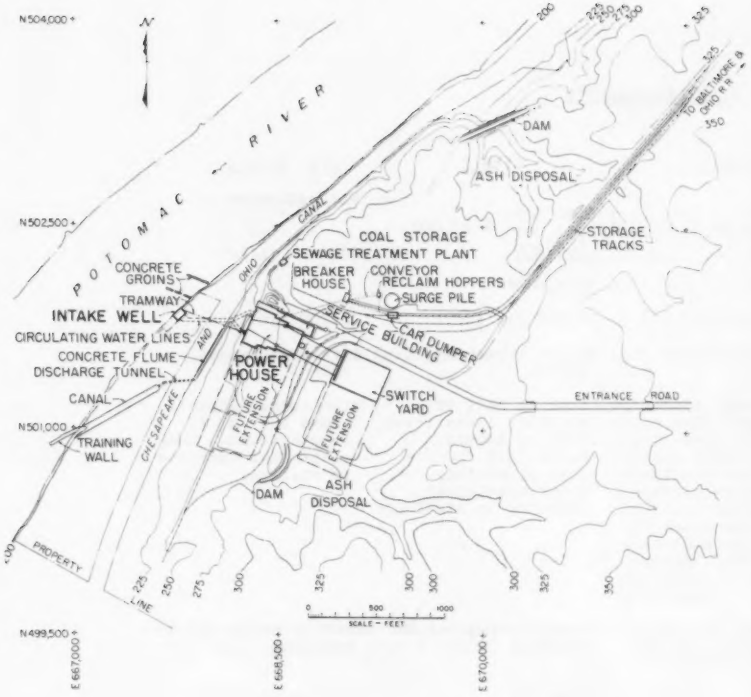


FIG. 4
SELECTED LOT PLAN

Circulating Water System

It was initially assumed that sufficient cooling water for at least two or, probably, three units could be diverted from the river at all times without appreciable recirculation. Model tests have confirmed this opinion. It has been further assumed that, for units Nos. 3 and 4 and possible later units, both river intakes and cooling towers would be required to avoid serious recirculation and thermal pollution. Adequate space has been allocated for these possible structures. A separate circulating water system was provided for each unit, from the intake to a common discharge flume. Fig. 6 shows the arrangement of circulating water systems for which yard space was provided. No underground installations of any kind were placed where they would interfere with this future construction.

Intake and discharge tunnels were run double-barreled for convenience and economy. Each tunnel is formed by a 3/16 in. steel plate pipe encased in concrete. This double-barreled structure has been increased in size to provide space for 24 electrical conduits, service water, fire, and warm water recirculating lines. Where there are both intake and discharge bores, the electrical conduits are located on the cooler intake side of the structure. No individual underground service pipe lines and no overhead lines were

FIG 4A
AERIAL VIEW DURING CONSTRUCTION
JULY 1958

used at the low yard level in front of the station, to avoid possible flood damage.

Locations for the common discharge flume east or west of the canal, and at different elevations, were studied, with the arrangement shown in Fig. 4 and 6 being selected as the most economical.

Discharge from each unit enters the common discharge flume through openings in its bottom. A short distance downstream from each opening, the flume bottom is abruptly stepped to a lower level. This step, in effect, acts like a weir which serves to maintain the required submergence of the discharge outlets. The flume is of concrete in front of the station and forms base protection for a 20 ft high bank in the yard. It crosses under the C. & O. canal as a pair of concrete tunnels, but after emerging is converted into an open trapezoidal canal lined with dumped stone, which empties into the river approximately 1,100 ft downstream from the intake.

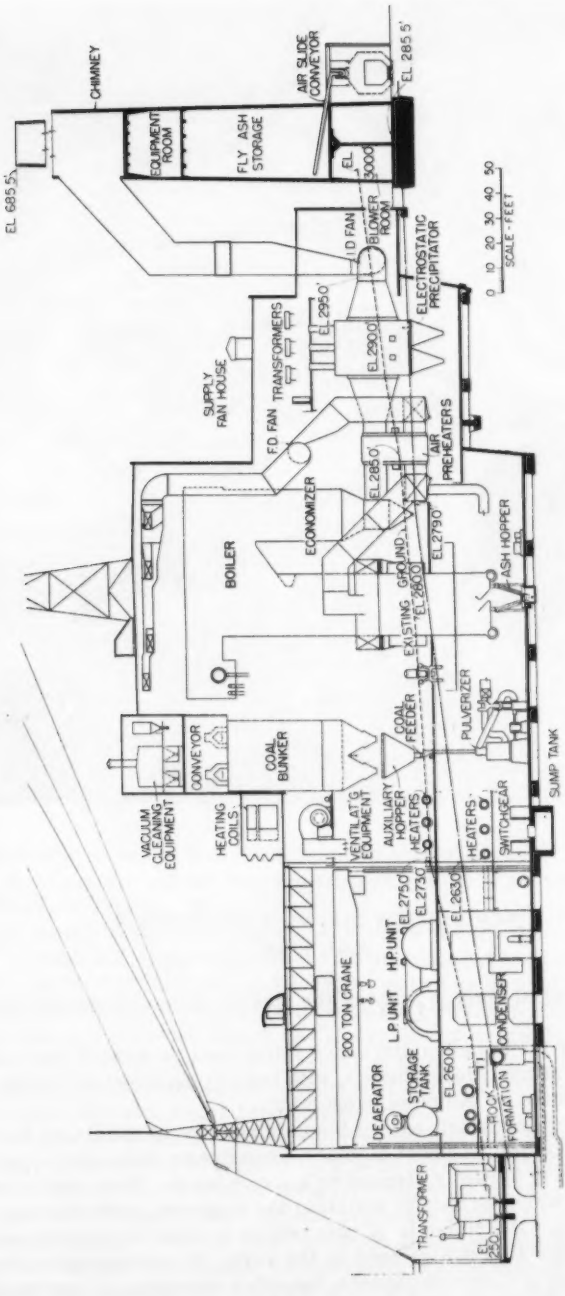


FIG. 5
CROSS SECTION OF POWER STATION

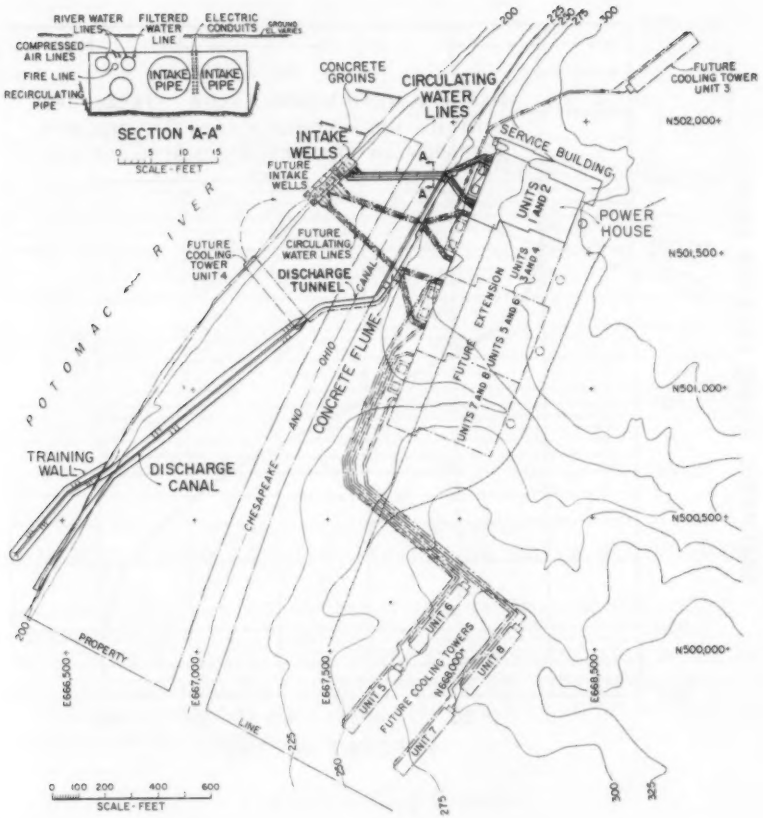


FIG. 6
PLAN OF CIRCULATING WATER SYSTEM

River Flow

The recorded rates of flow of the Potomac River at the plant site have varied from a minimum of 620 cfs to a maximum of 482,000 cfs with corresponding water levels of El. 190 and El. 230. The maximum 500 yr frequency flood predicted by the Corps of Engineers is 700,000 cfs, with a corresponding surface level of El. 242. At normal stages, the river is shallow, about 1,000 ft broad, and has a generally level, rocky bed.

The Corps of Engineers has proposed to construct a dam downstream at River Bend with crest at El. 235, which would result in a possible water surface of about El. 250 at the plant, based on the predicted maximum flood of 700,000 cfs. By agreement between Potomac Electric Power Company and the Corps of Engineers, all permanent installations at this site will be protected against water to El. 250.

A mean monthly flow-duration curve, showing water elevations for 0 to

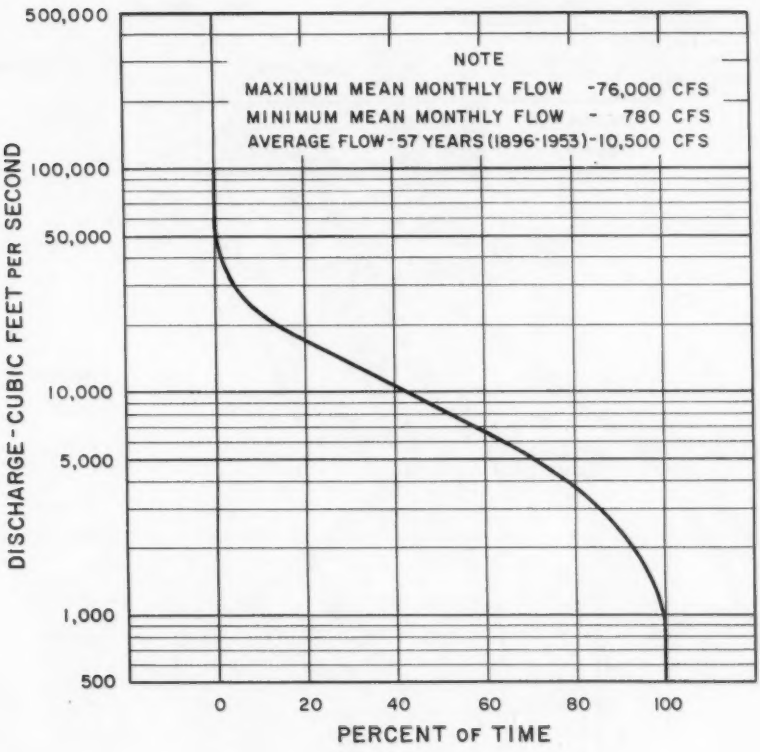


FIG. 7
FLOW - DURATION CURVE
MEAN MONTHLY

100 per cent of the time, appears in Fig. 7, and indicates that a wide range exists, from a minimum mean monthly flow of 780 cfs to a maximum mean monthly of 76,000 cfs, with an average flow of 10,500 cfs. However, daily flows are of more interest, particularly low ones which do not meet the requirements of up to four, six or eight units. However, these flows may occur for only a small part of the time.

The upper curve in Fig. 8 shows mean monthly flows recorded between 90 and 100 per cent of the time. The lower curve shows mean daily flows plotted from records for the range between 98 and 100 per cent of the time, extrapolated to 90 per cent of the time. The latter curve also shows that the minimum quantity of daily flow available for 100 per cent of the time is 620 cfs. Three 175,000 kw units require about 636 cfs. This theoretical scarcity has occurred one day in the 44 years of record. Whether this condition will be met by reduction of pumping, by recirculation of discharge water at loss in efficiency and with increased thermal pollution, or by purchasing power

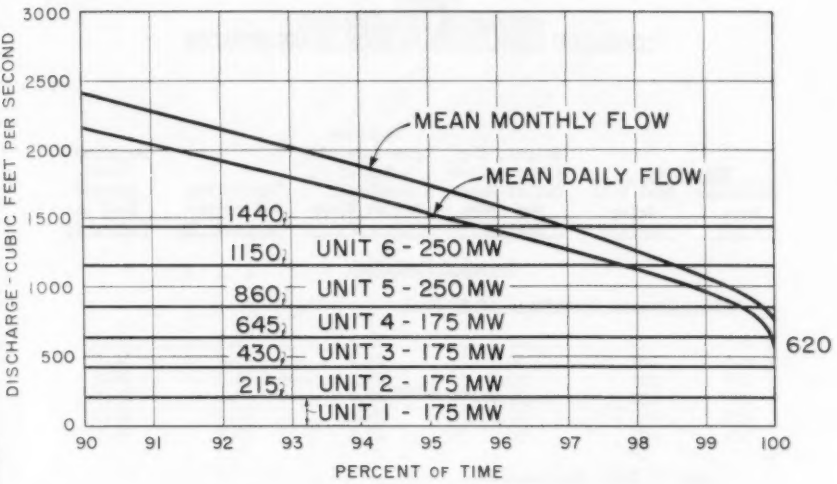


FIG. 8

FLOW-DURATION CURVE

MEAN MONTHLY AND DAILY VS CIRCULATING WATER REQUIREMENTS

from an interconnection with another system, can be decided when Unit No. 3 is authorized.

For Unit No. 4, full flow would be available 99.7 per cent of the time. If it is assumed that units Nos. 5 and 6 may be of 250,000 kw capacity each, full flow would be available 95.7 per cent of the time for Unit No. 6 or all but the equivalent of 16 days a year.

Condenser Floor Elevation

Elevations from 223 to 247 for the condenser room floor were considered possible by varying the floor levels in the boiler room area to minimize rock excavation. Table 2 shows comparative operating data for one-pump and two-pump operations. The required elevations of water seals at the discharge flume are shown for the various condenser room floor elevations. To permit starting design drawings and to make comparative estimates, El. 235 was selected as a base and differences in cost for other levels were listed, the results being shown in Table 3 and Fig. 9.

Variations in cost between El. 235 and El. 247 were insignificant for units Nos. 1 and 2, with El. 235 the lowest value. Variations in cost between El. 232 and El. 235 were small for units Nos. 3 and 4, with El. 235 the cheaper. El. 232 appeared to cost slightly less than El. 235 for units Nos. 5 and 6 considered alone and operating only on intakes for an assumed 90 per cent of the time. El. 235 proved to be the most economical elevation not only for units Nos. 1 and 2, but for units Nos. 1 through 4, and for units Nos. 1 through 6, when considered in these groupings and sequences of installation, as indicated by heavy summary lines on Fig. 9. El. 235 was retained for the final design.

TABLE 2
COMPARATIVE OPERATING DATA
CIRCULATING WATER SYSTEM ON RIVER INTAKE OPERATION

Elevation		Height, Top of Condenser Outlet Water Box Above Seal, Ft	Negative Pressure at Top of Condenser Outlet Water Box, Ft	Throttling Valve Loss in Head, Ft	Required Total Dynamic Head on Pumps, Ft
Condenser Room Floor	Seal Well Water Surface				
		Two-pump Operation			
Under Present Conditions of No Pool					
247	227	30.7	29.2	0.2	60.5
241	221	30.7	29.2	0.2	54.5
235	215	30.7	29.2	0.2	48.5
232	212	30.7	29.2	0.2	45.5
229	209	30.7	29.2	0.2	42.5
223	203	30.7	29.2	0.2	36.5
If Dam is Built to El. 235					
241	235	16.7	15.2	0.2	23.7
235	235	10.7	9.2	0.2	23.5
		Single Pump Operation			
Under Present Conditions of No Pool					
247	226.2	38.2	29.2	6.0	49.6
241	220.2	38.2	29.2	6.0	43.6
235	214.2	38.2	29.2	6.0	37.6
232	211.2	38.2	29.2	6.0	34.6
229	208.2	38.2	29.2	6.0	31.6
223	202.2	38.2	29.2	6.0	25.6
If Dam is Built to El. 235					
241	235.0	23.4	20.2	0.2	8.2
235	235.0	17.4	14.2	0.2	8.1

The costs of intake and cooling tower structures are not included in Table 3 and Fig. 9. Therefore, the comparisons show the differences in costs for varying floor elevations for each alternative arrangement, but do not show comparisons between the several arrangements for any given floor elevation.

River Water Intake Compared with Cooling Tower

To maintain station output at maximum capacity, cooling towers will be needed at some stage of development of the station. They can be arranged either to take over service temporarily from some intakes suffering from a temporary shortage of cool water, or to provide the entire circulating water requirements for such units. In the latter case, intakes could be omitted. Cooling towers on closed cycles may be used, or an open cycle may be used in which the hot water is taken from the open flume, cooled, and discharged into the river close to the intake, thus augmenting the supply of cool river

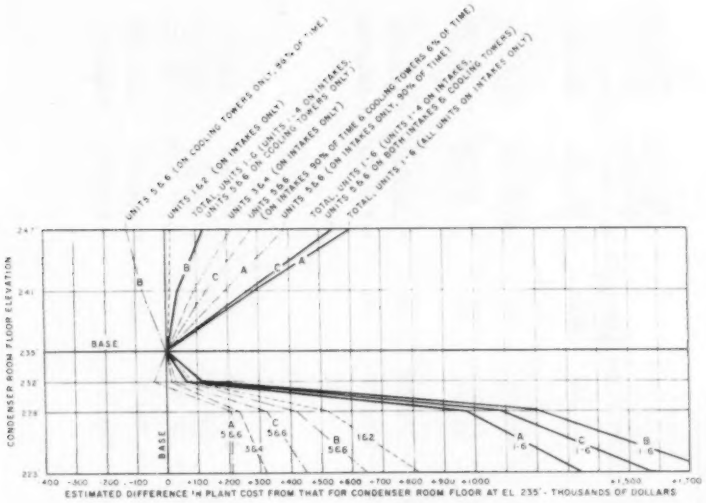

CONDENSER ROOM FLOOR ELEVATION
BASE
ESTIMATED DIFFERENCE IN PLANT COST FROM THAT FOR CONDENSER ROOM FLOOR AT EL 235' - THOUSANDS OF DOLLARS

FIG. 9
ESTIMATED CHANGES IN CONSTRUCTION AND CAPITALIZED OPERATION COSTS
FOR VARIOUS ELEVATIONS OF CONDENSER ROOM FLOOR

water and utilizing the intake well pumps to lift the water to the condensers. Studies of river flows and of capital and operating costs for intakes, towers, and circulating water systems developed information necessary to guide the plant layout for the initial step and future extensions.

River intake systems are approximately only one half as costly to install and operate as cooling tower systems at this particular site, partly because of the high elevations of available cooling tower locations that require high pumping heads, and partly owing to long penstocks for water lines in both directions. Another item unfavorably affecting cooling tower costs is an assumed life of 15 yr compared with 40 yr for a river intake. Thermal efficiency is also less with use of a cooling tower, except for a very small percentage of the time. The possibility also exists of installing both intakes and cooling towers, using the intakes, which are more efficient and less costly to operate, for as much of the time as possible, and the cooling towers only when lack of sufficient cool river water makes this necessary. For intermittent use of cooling towers, treated wood and fog sprays were included in the estimates. For these reasons, thorough consideration must be given to river flows and amounts and temperatures of recirculated water that can be obtained in the intake.

A study was made of four circulating water systems for Unit No. 4, shown in plan in Fig. 6 and in flow diagrams in Fig. 10 and 11. These arrangements are briefly described as follows:

Scheme I - River intake only, discharging to river by open flume

Scheme II - Cooling tower only, located north of the station, using a closed cycle requiring only make-up water. Sulfuric acid and polyphosphate treatments are required for maintenance but

TABLE 3
ESTIMATED CHANGES IN CONSTRUCTION AND CAPITALIZED OPERATING COSTS
FOR VARIATIONS IN CONDENSER FLOOR ELEVATION OF 235

	Condenser Floor Elevation					
	El. 223	El. 229	El. 232	El. 235 Base	El. 241	El. 247
Units 1 and 2 (Operate on Intake Only, 96% of Time)						
Excavation, building and underfloor pumping	+ $676,000	+ $473,000	+ $58,000	0	-$109,000	-$218,000
Circulating water piping, flumes and incremental pump costs	+ 330,000	+ 147,000	+ 69,000	0	+ 39,000	+ 67,000
Capability cost - $100 per kw, 40 kw per ft head	- 48,000	- 24,000	- 12,000	0	+ 24,000	+ 48,000
Capitalized generating costs - 40 kw x 365 days x 24 hr (= 351,000 kwhr) x 96% time x ft head diff. x $0.004/.13	- 124,000	- 62,000	- 31,000	0	+ 62,000	+ 124,000
Total, Units 1 and 2	+ $834,000	+ $534,000	+ $84,000	0	+ $16,000	+ $21,000
Units 3 and 4 (Operate on Intake Only, 96% of Time)						
Excavation, building and underfloor pumping	+ $429,000	+ $295,000	+ $47,000	0	- $35,000	- $38,000
Circulating water piping, flumes and incremental pump costs	+ 57,000	+ 26,000	+ 12,000	0	+ 45,000	+ 81,000
Capability cost - $100 per kw, 40 kw per ft head	- 48,000	- 24,000	- 12,000	0	+ 24,000	+ 48,000
Capitalized generating costs - 351,000 kwhr x 96% time x ft head diff. x $0.004/.13	- 124,000	- 62,000	- 31,000	0	+ 62,000	+ 124,000
Total, Units 3 and 4	+ $314,000	+ $235,000	+ $16,000	0	+ $96,000	+$215,000
Units 5 and 6 - Alternate A (Operate on Intake Only, 90% of Time)						
Excavation, building and underfloor pumping	+ $485,000	+ $342,000	+ $38,000	0	+ $8,000	+ $40,000
Circulating water piping, flumes and incremental pump costs	- 25,000	- 12,000	- 5,000	0	+ 48,000	+ 86,000
Capability cost - $100 per kw, 40 kw per ft head	- 48,000	- 24,000	- 12,000	0	+ 24,000	+ 48,000
Capitalized generating costs - 365 days x 24 hr (= 551,000 kwhr) x 90% time x ft head diff. x $0.004/.13	- 183,000	- 92,000	- 46,000	0	+ 92,000	+ 183,000
Total, Units 5 and 6 - Alternate A	+ $202,000	+ $201,000	- $32,000	0	+$185,000	+$384,000

Excavation, building and underfloor pumping	+ $485,000	+ $342,000	+ $38,000	0	+ $8,000	+ $40,000
Circulating water piping, flumes and incremental pump costs	- 25,000	- 12,000	- 5,000	0	+ 48,000	+ 86,000
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
(= 551,000 kwhr) x 90% time x ft head diff. x $0.004/.13	- 183,000	- 92,000	- 46,000	0	+ 92,000	+ 183,000
Total, Units 5 and 6 - Alternate A	+ $202,000	+ $201,000	- $32,000	0	+$185,000	+$384,000

Units 5 and 6 - Alternate B (Operate on Cooling Towers Only, 96% of Time)

Excavation, building and underfloor pumping	+ $485,000	+ $342,000	+ $38,000	0	+ $8,000	+ $40,000
Circulating water piping, flumes and incremental pump costs	+ 86,000	+ 43,000	+ 22,000	0	- 43,000	- 86,000
Capability cost - $100 per kw, 63 kw per ft head	+ 75,000	+ 37,000	+ 19,000	0	- 37,000	- 75,000
Capitalized generating costs	0	0	0	0	0	0
Total, Units 5 and 6 - Alternate B	+ $646,000	+ $422,000	+ $79,000	0	- $72,000	-$121,000

Units 5 and 6 - Alternate C (Operate on Intake 90% of Time and on Cooling Tower 6% of Time)

Excavation, building and underfloor pumping	+ $485,000	+ $342,000	+ $38,000	0	+ $8,000	+ $40,000
Circulating water piping, flumes and incremental pump costs	+ 61,000	+ 31,000	+ 15,000	0	+ 5,000	0
Capability cost - $100 per kw, 63 kw per ft head	+ 75,000	+ 37,000	+ 19,000	0	+ 37,000	+ 75,000
Capitalized generating costs						
Intake operation - 551,000 kwhr x 90% time x ft head diff. x $0.004/.13	- 183,000	- 92,000	- 46,000	0	+ 92,000	+ 183,000
Cooling tower operation	0	0	0	0	0	0
Total, Units 5 and 6 - Alternate C	+ $438,000	+ $318,000	+ $26,000	0	+$142,000	+$298,000

Total, Units 1-4, and 5-6 - Alternate A	+$1,350,000	+ $970,000	+ $68,000	0	+$297,000	+$620,000
Total, Units 1-4, and 5-6 - Alternate B	+$1,794,000	+$1,191,000	+$179,000	0	+ $40,000	+$115,000
Total, Units 1-4, and 5-6 - Alternate C	+$1,586,000	+$1,087,000	+$126,000	0	+$254,000	+$534,000

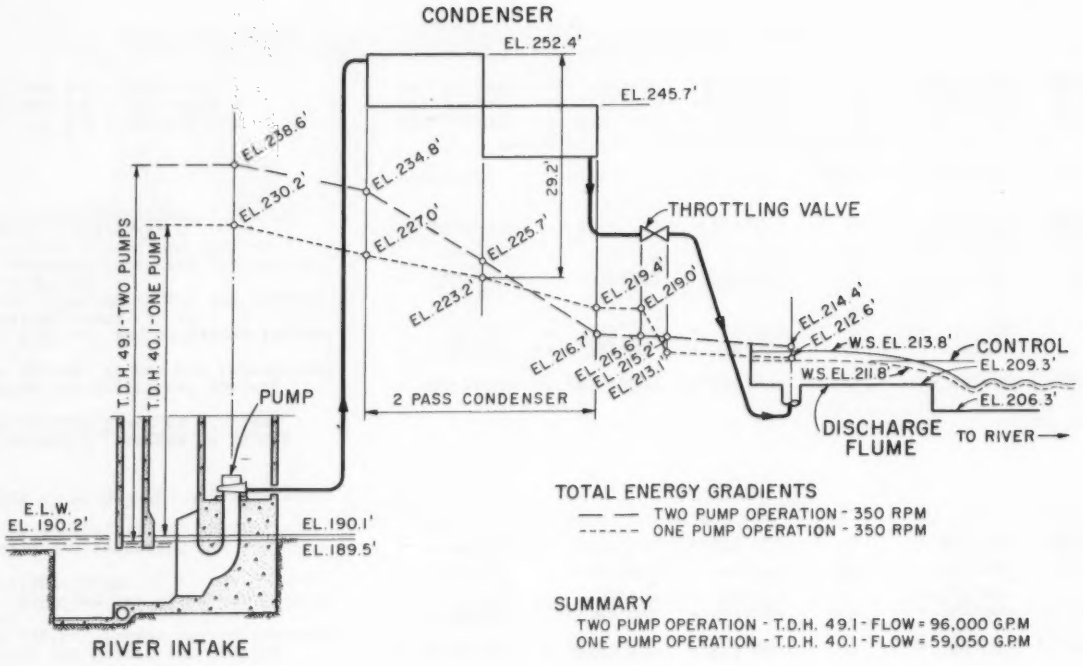
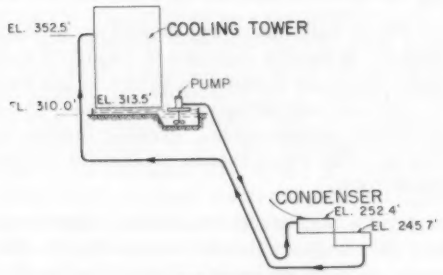
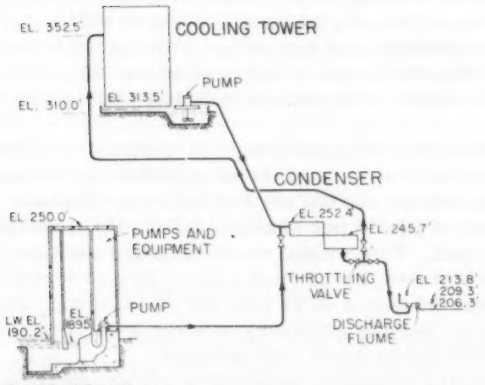


FIG. 10

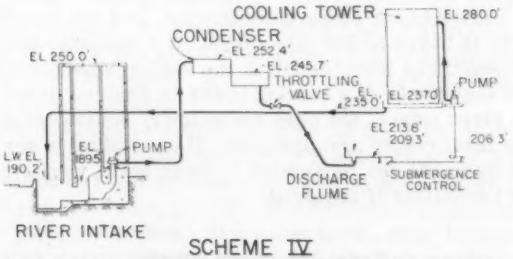
COOLING WATER FLOW DIAGRAM - RIVER INTAKE SYSTEM
SCHEME I



SCHEME II
COOLING TOWER CLOSED CYCLE



SCHEME III
RIVER INTAKE WITH CLOSED CYCLE COOLING TOWER
NORTH OF STATION



SCHEME IV
RIVER INTAKE WITH OPEN CYCLE COOLING TOWER
SOUTHWEST OF STATION

FIG 11
ALTERNATIVE COOLING WATER FLOW-DIAGRAMS
SCHEMES II III AND IV

no fog spray. The lumber in the tower need not be treated.

Scheme III - River intake with stand-by, closed cycle, cooling tower north of station. Butterfly valves are required to change the flow of water from the intake and flume to the cooling tower. Sulfuric acid and polyphosphate treatments and fog spray are required for maintenance. Treated lumber is required in the tower. The river intake is inoperative when river water is insufficient.

Scheme IV - River intake with stand-by cooling tower southwest of the station on a support located above the El. 230 flood level, designed as an open cycle. Prevailing winds blow under as well as over the tower. The tower draws hot water from the common discharge from all units in the flume to the cooling tower above, and discharges cooled water directly into the river slightly downstream from the intake, thus augmenting the supply of cool water. Sulfuric acid and polyphosphate treatments and fog spray are required for maintenance. Treated lumber is required in the tower. The tower is to be removed if the dam is built and a pool formed.

A comparative cost analysis of these four schemes is shown in Table 4. In addition to construction costs of the elements of the circulating water systems, a capability charge of $100 per kilowatt was allocated to the motor loads and a charge of $0.004 per kilowatthour for the time operated was capitalized at 13 per cent. The present worth of future charges for cooling tower maintenance was included, based upon a 15 yr rate of depreciation of a cooling tower as compared with a 40 yr rate for other items. The monetary effect of difference in thermal efficiency between water from cooling tower discharge and river intake water was included.

These comparative estimates indicate the greatly increased cost of installing and operating a cooling tower either alone or in conjunction with a river intake and give bases from which the power company can consider the price that could be paid for a physical interconnection with another power source plus power charges. An interconnection has the added advantage of making power available for other emergencies, and not merely firming the system capacity at times of low river flow. If a cooling tower is to be installed, the closed cycle tower north of the station appears to be the most economical arrangement. If a cooling tower is desired to act only as a supplement to the river intake, the open cycle tower southwest of the station appears to be the least costly arrangement. If no cooling tower is installed for Unit No. 4, the estimate can be applied to later units, modifying for the effects of larger capacities if required.

Hydraulic Model Studies

Hydraulic model tests were made at the Alden Hydraulic Laboratory of Worcester Polytechnic Institute, to determine if cooling water flow for three and four units could be diverted from the river during extreme low water without a dam or submerged weir across the river width, to study hot water flow patterns caused by the condenser discharge, avoid silting conditions,

and test the hot water discharge location and flume construction.

While the laboratory's final report was not available at the time of these studies, preliminary information indicated that no dam or weir would be required across the river at this site for the first two units now under construction. The assumption that sufficient river water would always be available for three units was tentatively confirmed, and a supply for the fourth unit also appeared to be present, if a small amount of recirculation is permitted at the rare times of extreme low flows. At such times, when the pumping rate exceeds the river flow, recirculation will take place, but the model studies indicate that the temperature of the recirculated water will be lowered by evaporative cooling, and that for stabilized conditions, the temperature rise at the intake will be only a few degrees. However, at such times of low flow, if the natural river water temperature is high, the flume discharge may be so hot that thermal pollution of the river may occur, in which case a cooling tower might be desirable.

Preliminary information from the model tests indicated that lowering of the river rock bed for some distance in front of the intakes permits diversion of the river flow into the intake wells, and that at low water periods, with some recirculation, the equivalent of full river discharge will be available. In order to prevent silting of this depressed area at periods of high flow, a series of the tests were performed, which indicated that low, diagonally placed groins should be provided upstream from the intake structure. To prevent accumulation of floating trash in front of the intake, a log boom will be provided, running diagonally from the outboard end of the downstream groin to the downstream corner of the intake.

Access to Intake Wells

The intake well structure rests on rock, with the surrounding yard at El. 215 and an operating floor at El. 250. The land upon which it rests is separated from the main property by the C. & O. canal. Intercommunication between the power station and the intake house should be maintained at all times. Various tunnel, bridge and tramway schemes are shown in Fig. 12. Any crossing of the canal must be by tunnel, or by a bridge with boat clearance above nominal canal water level of El. 210, since canal boat traffic may be revived for park visitors. A low level bridge should also clear the flood of record, El. 230. If designed to withstand the force of large floating objects, the cost would be increased. If a dam is built downstream, this low-level bridge would be submerged.

A flooded pool would require a high bridge with deck no lower than El. 250, and piers and deck should be designed to resist impact of a floating tree or house and the water current pressure on such objects, spans and piers. High level El. 250 and low level, El. 234, foot bridges 6 ft wide and truck bridges 16 ft wide were estimated. At the low level, after bridging the canal, alternative estimates were made for supported spans or roadway on embankment to the intake structure.

Another possibility would be a tunnel, constructed at the side of the intake tunnel, for which 11 ft width and 7 ft height were assumed. Conduits and service pipes would be racked on the walls. A motorized unit would haul a dolly for loads to be carried through the tunnel. The tunnel floor would be at El. 199 to pass under the canal bed, and, since transportation by dolly

TABLE 4
COMPARATIVE APPROXIMATE ESTIMATES
COOLING TOWER VS. RIVER INTAKE FOR UNIT 4
FOR ESTABLISHED FLOOR ELEVATION 235

	Scheme I	Scheme II	Scheme III	Scheme IV
	River Intake System	Cooling Tower System, Closed Cycle Tower North of Station	River Intake With Closed Cycle Cooling Tower North of Station	River Intake With Open Cycle Cooling Tower Southwest of Station
River Intake System				
Intake structure	$441,000		$441,000	$441,000
Mechanical equipment	159,000		159,000	159,000
Electrical equipment	204,000		204,000	204,000
Intake and discharge lines and flume	248,000		277,000	277,000
Cooling Tower System				
Cooling tower		$501,000	548,000	548,000
Cooling tower support		100,000	100,000	327,000
Pumps and piping		907,000	907,000	164,000
Electrical equipment		403,000	403,000	415,000
Condenser water boxes, additional		26,000	26,000	
Water Treatment Equipment	8,000	21,000	21,000	8,000
Capability Charges (@ $100 Per Kw)				
River intake circulating water pumps(a) - 48.5 ft total dynamic head	97,000			
Cooling tower circulating water pumps				
North of station - 75 ft TDH		150,000	150,000	
Southwest of station - 80 ft TDH				160,000
Cooling tower fans(b)		88,000	88,000	88,000
Capitalized Operating Costs				
River intake circulating water pumps(d) - 48.5 ft TDH	225,000		223,000	223,000

North of station - 75 ft TDH		150,000	150,000	
Southwest of station - 80 ft TDH				160,000
Cooling tower fans(b)		88,000	88,000	88,000
Capitalized Operating Costs				
River intake circulating water pumps(d) - 48.5 ft TDH	225,000 86%(h)		223,000 85.5%(h)	223,000 85.5%(h)
Cooling tower circulating water pumps				
North of station(d)- 75 ft TDH		348,000 86%(h)	2,000 0.5%(h)	
Southwest of station(d)- 80 ft TDH				2,000 0.5%(h)
Cooling tower fans(e)		203,000 86%(h)	1,000 0.5%(h)	1,000 0.5%(h)
Cooling tower fog spray pumps(f)			13,000	13,000
Other Considerations				
Present worth of fixed charges for rate of depreciation excess for cooling towers over river intakes(g)		314,000	343,000	343,000
Lower average thermal efficiency				
When operating on river intake	6,000			
When operating on cooling tower		220,000		
Total Comparative Costs	$1,428,000	$3,281,000	$3,906,000	$3,470,000

Footnotes:

(a) 90,000 gpm x (8.33/33,000) x (.75/.85 efficiency) = 20 kw per ft of total dynamic head.

(b) 990 bhp x .75/.85 efficiency = 875 kw.

(c) Allowance due to possible river water temperature excess of 2 F over cooling tower discharge on hottest day at 200 kw per deg F and $100 per kw.

(d) 365 days x 24 hr x 20 kw x TDH x ($0.004/.13) x time factor.

(e) 365 days x 24 hr x 875 kw x ($0.004/.13) x time factor.

(f) 320 gpm x (8.33/33,000) x (.75/.85 efficiency) x 365 days x 24 hr x ($0.004/.13) x 50 ft head x (1 - time factor).

(g) Assumed 15 yr life of cooling tower and 40 yr life of river intake. (6.67% - 2.50%) x 15.046 present worth factor x estimated cost of tower.

(h) Time factor. Predicted equivalent uniform per cent of total time that auxiliaries are operated, on 6% present worth basis, on 40 year life.

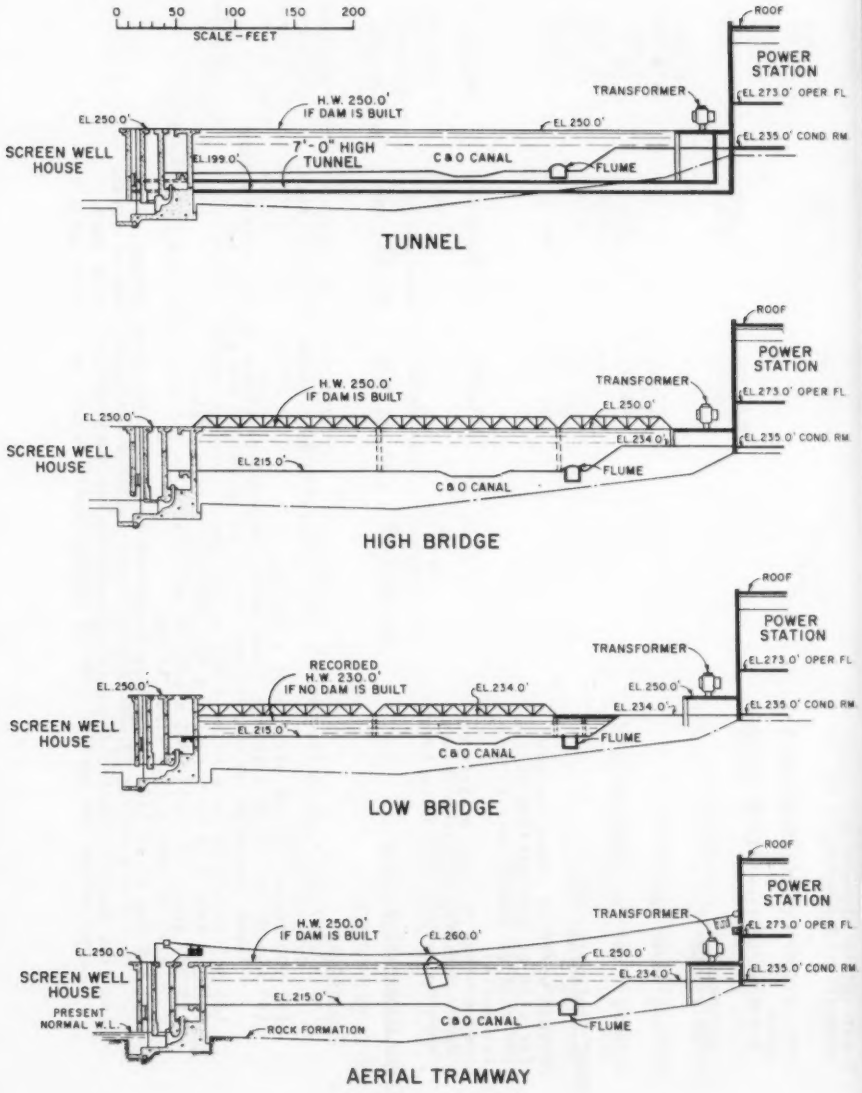


FIG. 12

ALTERNATIVE METHODS FOR REACHING SCREENWELL HOUSE

requires a level floor, deep shafts would be needed at both ends, with stairs, man lifts and hoists. Ventilation would also be required.

Another means of access would be an aerial tramway, connecting the main turbine room operating floor at El. 273 with the top level of the intake well at El. 250, for which a cab having a live load capacity of 750 lb would be provided. Cable clearance is at El. 260. The hauling cable would be pulled from an electric drive.

Table 5 shows estimated costs for variable items, and the net excess cost chargeable to access after subtracting the cost of placing service pipes, conduits and cables in the circulating water tunnel wall, as required if no means of access is provided.

The tunnel would not be damaged by floods and would, therefore, always be available, but provides an inconvenient route and is most expensive. All truck bridge schemes would be costly and the structures subject to flood damage. Some saving would result from using foot bridges, but heavy equipment would have to be ferried or swung over the canal, thus making the cost of providing man-access considerable. If a low-level bridge should be built and the dam later constructed, the investment in the low bridge would be largely lost and other means of access would then need to be provided.

The aerial tramway was selected because of low cost, safety against floods, availability under all conditions, and convenience for personnel going between the service building and intake. Heavy equipment can be lifted or ferried across the canal.

Station Description

The power plant building is being constructed in the first stage to house units Nos. 1 and 2. It consists of a turbine room, auxiliary bay, bunker bay, boiler room, precipitator bay and fan bay. The condenser floor level at El. 235 is carried through the boiler area, stepped up to El. 260 under the air heaters and precipitators, then to El. 284 in the induced draft fan bay.

Reinforced concrete footings are poured on rock and designed for a bearing value of 10 tons per sq ft. Under the floor slab at El. 235 rock was excavated to El. 231, and the slab poured on granular fill. A collecting system was installed below the floor slab to remove infiltration, especially if the dam is constructed downstream and a pool level of El. 235 and a flood level of El. 250 occur. The collected water will be drained to a sump and pumped out. Permanent base walls are reinforced concrete up to El. 250. The temporary end wall is reinforced concrete up to El. 242 which later could be extended to El. 250.

The building frame is structural steel. The turbine room floor is reinforced concrete, other areas are galvanized grating. Siding is corrugated asbestos, except that asbestos protected metal is used for trim and for protection of the building adjacent to the main transformers. Sash is galvanized steel. The roof is tar and gravel over precast concrete channel slabs.

Two 3,500 lb capacity, 200 fpm speed freight and passenger elevators are installed in the boiler room.

Two unlined bunkers are constructed of Mayari-R steel, and will hold a total of 5,700 tons. A single chimney, 400 ft-0 in. high by 16 ft-6 in. top inside diameter, serves the two boilers. It is built of reinforced concrete, with corbeled brick lining, supported on a concrete mat on rock. The lower

ESTIMATED
FOR ACCESS T

Scheme	I	II	III
	Aerial Tramway Clearing El. 260	Foo (6 Not to Floati Low Leve El. 23	ridges Wide) istant rge Objec High El.
Tramway			
Structural supports	$6,000		
Equipment	10,000		
Electrical connections	2,000		
Bridge			
Span		$16,000	$16
Abutments and piers		5,000	10
Embankments, riprap, road surfacing, and guard rails			
Intake House Extension			
Stairs	2,000	2,000	
Service pipe encasements in circulating water tunnel walls	26,000	26,000	26
Conduit encasement in circulating water intake tunnel walls	3,000	3,000	3
Electrical			
Manholes	14,000	14,000	14
Conduit supports and brackets			
Conduit and cable, including trays	111,000	111,000	111
Lighting	1,000	6,000	6
Tunnel			
Structure and end shafts			
Lengthen flumes			
Elevators			
Hoist			
Tractor			
Pipe supports			
Heating and ventilating			
Total	$175,000	$183,000	$186
Sum chargeable to intake access	$26,000	$34,000	$37
Remarks			
Additional cost for embankment instead of trestle			
Decrease cost if conduit carried on bridge			

*Indicates items re red i

BLE 5
MPARATIVE COSTS
RIVER INTAKE WELLS

II	III	IV	V	VI	VII
Foo	ridges				
(6	Wide)	Truck Bridges (16 Ft Wide)			
Not R	istant	Not Resistant	Resistant		
to	rge	to Large	to Large		
Floati	Objects	Floating Objects	Floating Objects		
ow Leve	High Level	Low Level	Low Level	High Level	Tunnel
El. 234	El. 250	El. 234	El. 234	El. 250	El. 199
$16,000	$16,000	$74,000	$90,000	$88,000	
5,000	10,000	29,000	47,000	66,000	
				41,000	
					$14,000
2,000		2,000	2,000		
26,000	26,000	26,000	26,000	26,000*	
3,000	3,000	3,000	3,000	3,000*	
14,000	14,000	13,000	13,000	13,000*	
			11,000	11,000	
111,000	111,000	111,000	103,000	107,000*	93,000
6,000	6,000	6,000	6,000	6,000	6,000
					273,000
					9,000
					45,000
					7,000
					7,000
					4,000
					4,000
$183,000	$186,000	$264,000	$301,000	$361,000	$462,000
$34,000	$37,000	$115,000	$252,000	$212,000	$313,000
			$72,000		
				$8,000	

items re red if no access provided.

portion is used as a silo for 800 tons of fly ash, which can be discharged into trucks or railroad cars.

Service Building

The service building arrangement, shown in longitudinal section in Fig. 13, is governed by accessibility of the individual services and by the topography. It is built against the north wall of the station. It contains a variable number of stories, being stepped to conform to the natural rock slope and the terrain. It is 60 ft wide by 435 ft long, with seven floor levels from El. 235 to El. 293.

The warehouse floor, at the east end, at El. 289 is 4 ft above adjacent yard level and is located near the spur track. In the middle section, a records room at El. 293 is underlaid by the shops at El. 273, where access is available both to the station operating level and to the parking area. In the west portion, at El. 285, rooms are located for the laboratory, emergency power dispatcher, ventilation equipment, and future expansion. At El. 273, offices and locker rooms are located in the west portion, and shops in the middle portion, opening directly onto the station main operating floor. In the west portion, water treatment equipment is located at El. 262 and El. 250, connecting to these levels in the main station. The lube oil room and lower portions of the clear wells are located below, at El. 235 and open onto the condenser room floor. Space allocations for the service building are shown in Table 6.

The service building has structural steel framing, green asphalt protected metal walls, and precast concrete channel roof slabs. The roof over air-conditioned areas is insulated.

The parking area for 71 cars is on a slight slope from El. 285 at the start to El. 273 at entrances to the offices and shops. A road leads off to circle into the chemical storage room at El. 262, and also branches into a road to the sewage treatment plant.

An 11,000 lb capacity freight elevator connects the condenser room and main operating floors with shop, office and laboratory floors of the service building. The adjacent service elevator in the boiler room also connects to upper levels of the main station.

By adapting the service building, parking, and road layouts to the terrain, economy was realized, and the various components of the service building were placed at the most convenient elevations for road access and for plant operations.

CONCLUDING REMARKS

Pressure of the construction schedule required assumption of some decisions before the analyses could be completed. This was the case for selection by the Owner of El. 235 for the condenser room floor; this gratifyingly proved to be a correct choice. The station setback from the C. & O. canal had to be selected at an early date, and space provided for penstocks for cooling towers; completed analyses confirmed the desirability of the arrangement.

The studies were made from a civil engineering viewpoint for the purpose of securing a lot plan arrangement and building elevation that would

FIG. 13

LONGITUDINAL SECTION-SERVICE BUILDING

TABLE 6
SPACE ALLOCATIONS IN SERVICE BUILDING

Elevation	Item	Area, Sq Ft	Sq Ft
293	Company records storage	5,750	5,750
289	Storage warehouse	10,600	10,600
285	Blueprints and records	450	
285	Laboratory	1,400	
285	Coal preparation room	120	
285	Emergency power dispatcher	1,160	
285	Ventilation equipment room	810	3,940
273	Machine shop	5,700	
273	Administration	2,800	
273	Toilet and locker rooms	4,100	
273	Tool room	1,020	
273	Test shop	780	
273	Electrical shop	900	15,300
262	Water treatment	6,900	
250	Water treatment	8,500	
235	Water treatment	1,450	16,850
235	Lube oil room	1,320	1,320
	Total, Initial Construction		53,760
285	Future expansion of record storage or laboratory	4,200	4,200
289	Future expansion of storage warehouse	7,500	7,500

accommodate future eventualities in a convenient and economical manner, without foreseeable interferences or undue construction difficulties.

Availability of volumes of river water at the intake, equal to the total river flow, was assumed, and appears to be confirmed by preliminary reports of hydraulic model tests. Fortunately, no corrective measures were required in the design assumptions as a result of preliminary tests and completion of analyses.

The economic value of such studies may be seen from inspection of the estimated cost differentials for comparative schemes. Presentation of future relative estimated costs for expansions permits consideration of the relative ultimate costs.

Studies of cooling tower and river intake cost differentials will be of assistance in choosing between this and other sites for future units, and in consideration of the economics of expansion versus interconnection with other systems. Studies of open and closed circulating water systems allowed sufficient spaces to be provided on the lot plan for various arrangements.

The relative costs found for this project, and the conclusions drawn, would not apply under different conditions. Power plant sites are apt to vary widely. The types of investigations made, however, and the methods of approach to the problems, may be of value in other instances.

The project is being designed and constructed by Stone & Webster Engineering Corporation under the general direction of Mr. L. W. Cadwallader, Vice President and Mechanical Engineer, Potomac Electric Power Company.

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MULTI-LAYER PENSTOCKS AND HIGH PRESSURE WYES[a]

Closure by Ewald Schmitz

EWALD SCHMITZ.[1]—The writer is grateful to Messrs. S. V. Williams, F. Heidinger and C. S. Ospina for their contribution to the discussion.

Mr. Williams' reference to the ship-plate-structure committee's findings relative to the improved metallurgical condition of thin plates, particularly with respect to resistance to impact loadings and superior resistance to brittle fracture, is well taken.

In answer to Mr. Heidinger's request, the wyes for the Kemano & Cubatao penstocks were partially assembled on positioners. The components were preheated to 200° to 300° F and welded using low hydrogen electrodes. A method of block welding or alternating intermittant welding of the joining parts was used followed by intermediate peening. No thermal stress relief was required until completion of the welding operation.

Mr. Ospina asks about the purpose of the vent holes. In a multi-layer vessel only the innermost layer is pressure tight; the load-bearing layers are vented to the outside protecting the outer layers from excessive pressure buildup in the event of leakage of the inner shell thus providing an automatic safety and warning device. In other applications of multi-layer pressure vessels involving hydrogenation, venting prevents embrittling attack on the loadbearing layers.

In his second question Mr. Ospina implied that the manifold for the Cubatao underground plant failed during testing and asked what lessons were learned from the failure. The writer is unaware of any difficulty during testing. The Chief Mechanical Engineer and Chief Hydraulic Engineer of Cobast have stated that they had experienced no difficulty. The manifold was encased in concrete after welding without further test and has been in trouble-free operation under full load ever since.

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a. Proc. Paper 1344, August, 1957, by Ewald Schmitz.

1. Chf., Engr., Specialty Products, Process Equipment Div., A. O. Smith Corp., Milwaukee, Wis.

DESIGN OF LARGE PRESSURE CONDUITS IN ROCK[a]

Closure by F. W. Patterson, R. L. Clinch and I. W. McCaig

F. W. PATTERSON,[1] R. L. CLINCH,[2] and I. W. McCAIG.[3]—The authors were interested to note that all the contributors to the discussion were in substantial agreement with them as to the main problems in pressure conduit design.

The decision as to whether a steel liner is required or whether the water pressures can safely be applied to the overlying rock should be made only after careful consideration of stresses in the surrounding rock. Mr. Lawton points out that when calculating these stresses, consideration should be given to Heim's work on rock stresses at depth. Heim considered the rock to be plastic so that $\sigma_H = \sigma_V = wz$. Terzaghi and Richart's work can be considered as an extension of Heim's insofar as they also consider what the rock stresses will be adjacent to a cavity if σ_H is not equal to σ_V but only a fraction of it.

The authors regret the error in the section dealing with rock stresses due to water pressures in an unlined conduit. The term p/2wz should indeed be 2wz/p as Mr. Lawton suggests.

Mr. Bleifuss has stressed the fact that the structural safety of a steel-lined pressure conduit is dependent on the proper placing of the concrete around the liner. The authors agree with Mr. Bleifuss that grouting is essentially a blind operation and that the development of an economical and reliable expanding concrete would greatly reduce the gap which forms between the steel liner and the concrete while the concrete is curing. They also agree that even if complete tightness is attained during construction, this does not ensure freedom from gap formation after the initial filling.

Mr. Bleifuss has inquired whether field observations of stress distribution were made on the installation described. Measurements were not performed at Bersimis No. 1 and no provision has been made to carry out such observations at the other installations.

All contributors to the discussion agree that strength against external pressure is an essential factor in the design of steel conduit liners. Mr. Danel and Mr. Rueff drew attention to a series of experiments which gave values somewhat greater than those obtained by Amstutz' formula published in 1953 but showed that, for a diameter-to-thickness ratio of 273, this difference is not great. They also note that the use of stiffening rings greatly increases the strength of the liner against external pressure. The design thickness of 25/32 inches under a static head of 265 feet used at Picote

a. Proc. Paper 1457, December, 1957, by F. W. Patterson, R. L. Clinch and I. W. McCaig.

1. Sr. Engr., Hydr. Div., H. G. Acres & Co. Limited, Niagara Falls, Canada.
2. Sr. Project Engr., Hydr. Div., H. G. Acres & Co. Limited, Niagara Falls, Canada.
3. Hydr. Engr., H. G. Acres & Co. Ltd., Niagara Falls, Canada.

shows that steel stresses due to internal pressure without rock support were kept to a value commonly employed on exposed penstocks. The authors have rechecked the curve given in Figure 4 against Amstutz' 1953 formula and find it correct for a yield stress of 32,000 psi and a Young's modulus of 30 x 10^6 psi. Mr. Danel and Mr. Rueff may have used other values in their calculations.

Mr. Lawton appears to have misinterpreted Figure 4; no buckling occurred during the grouting of the Bersimis No. 1 steel lined pressure conduit. Mr. Lawton also requested clarification of the method of filling the penstocks and supply tunnel. This was accomplished by releasing a definite quantity of water from the intake rather than by pumping from the tailrace.

THE HAAS HYDROELECTRIC POWER PROJECT[a]

Closure by J. Barry Cooke

J. BARRY COOKE,[1] M. ASCE.—A number of important features have been emphasized by Mr. Lawton's discussion and some questions have been raised. The writer will first respond to the discussion and then briefly present some data regarding the later stages of construction and operation since the writing of the paper.

Galvanizing has been used extensively in the Sierras for ladders, transmission towers and other structures exposed to the weather. The life of the coating is of long and indefinite length. However, underwater the use of galvanizing is satisfactory in some watersheds and not in others. On the Mokelumne River, galvanized steel installed on various structures in 1931 is now badly corroded underwater and in perfect condition where only exposed to the weather. On the Kings River, a galvanized trash rack installed in 1927 is now only partially tuberculated and it was decided to galvanize the new underwater trash rack on that river. Unless there was experience to show underwater galvanizing to be satisfactory, the writer would use the more customary coal tar or asphalt coatings.

A number of submerged oil cylinder gate operators have been used by P. G. and E. in the last ten years. The piston shaft is, of course, of stainless steel. There have been no corrosion problems to date. Where the cylinder may not be used for many years, the packing gland end of cylinder and the stainless steel shaft have been packed in grease and wrapped with burlap.

It is the writer's opinion that comparisons of Manning's "n" between tunnels with and without lined invert cannot reasonably be made on the basis of measured head losses of different tunnels, because of the variation in overbreak and roughness of the rock. It is believed that a theoretical analysis, such as made for Haas, is necessary even though head losses for unlined tunnels with and without lined inverts are available.

The capacity of the Haas storage sand trap is based on an estimate that the 5 fps velocity on the design area will not move more than the top 2.2 inches of the 12 to 15-inch depth of muck. The largest material moved is estimated to be less than 2 inch and close to 1-1/2 inch in dimension. The difference in size and amount of material moved from an unmucked invert varies greatly between velocities of 4.0 to 6.5 fps.

Cracks similar to those which occurred at Haas wye-branch have also occurred on the "C-Clamps" of many other wye-branches. The cracks on the Haas wye were more extensive due to the welding sequence required by the delivery schedule. Where the pant-leg is welded to the side of the C-Clamp, tension due to shrinkage from the welding occurs across the C-Clamp

a. Proc. Paper 1529, February, 1958, by J. Barry Cooke.

1. Supervising Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

plate. The tensile strength across the thick C-Clamp plate is sometimes inadequate to resist the stresses, and cracks in the plane of the laminations of the rolled C-Clamp plate occur. One means of avoiding this is to make the intersection of C-Clamp and pant-legs without welding the pant-legs to the side of the C-Clamp, but to a forged round member. This would avoid the poor detail of taking stresses across the plane of rolling the C-Clamp plate at the cost of more welding.

The Perfo method of the Sika Chemical Corporation was adopted for the powerhouse chamber roof bolting on the basis of being a very satisfactory and minimum cost method of obtaining a fully grouted overhead anchor. It costs a little more than an ungrouted split-wedge rock anchor, but definitely less than grouted overhead split-wedge rock anchors, and grouting was considered to be required to prevent corrosion and assure permanence. The rock was of such quality that tensioning of the anchors was not necessary. The Perfo method is adaptable to simple production work and the anchors went in very rapidly.

The rockfill dams were both completed in 1958, one season ahead of schedule, and the units began operation in December 1958. The Wishon and Courtright Dams, and the earlier P. G. and E. Company rockfill dams have been discussed in two recent papers.(1,2) Wishon Dam was filled rapidly, in 13 days, in May 1958 and vertical crest settlement for the maximum section of the Main and Wing Sections has been:

	VERTICAL SETTLEMENT			
	Main Section Ht. 255 Ft.		Wing Section Ht. 150 Ft.	
	Ft.	% Ht.	Ft.	% Ht.
13 days filling + 9 days	1.31	0.52	0.40	0.27
Next 54 days full	.37	0.14	0.12	0.08
Next 77 days full	.04	0.01	0.02	0.01
Filling + 140 days	1.72	0.67	0.54	0.36

The main settlement is seen to take place during and just after filling and the magnitude is very comparable to that of the Lower Bear River Dams.(1) The Howell-Bunger valve installation was tested and operated successfully. When operating during freezing weather, the cold air supply to the air vents makes it necessary to maintain flow in the water motor supply lines or to drain them in order to prevent freezing.

Excluding the 645 feet of steel-lined section at the downstream portal, the 32,000-foot long tunnel is unlined for 97% of its length. They are 10 lengths of concrete lined sections about 50 to 150 feet long each, totaling 900 feet, and 4 lengths of gunite lining of 10 to 50 feet length each, totaling 120 feet. The gunite lining was used in the faulted and fractured zones that did not contain much soft-gouge material. The concrete lining was used in faulted zones containing zones of soft material. Gunite was used wherever it was considered acceptable for the short lined sections in order to minimize transition head losses. The larger area of cross section of the gunite lined section has lower velocities and lower transition losses. A small amount of concrete and gunite work was done to soft seams where adjacent rock was not fractured or seamy. Steel sets were removed in two locations where they were considered necessary only for safety during driving.

The tunnel was filled on November 28, 1958 and at that time the Wishon Reservoir was still full, awaiting initial operation of Haas Power House in December. Immediately after filling, the intake butterfly valve was closed and a leakage check was made by recording the water surface in the 50-foot diameter concrete surge tank. The leakage was 3 cfs. Four days later another check was made and leakage was 0.6 cfs. Such small leakage for a newly-filled six and one-quarter mile unlined tunnel, under 240 to 330 feet of head, is very satisfactory. The leakage measurement is very accurate since the penstock butterfly valve and intake butterfly valve were both drip tight. The inflow leakage during construction did not exceed 0.7 cfs. The average overbreak was 21.9% which represents an overbreak of 0.71 feet all around the 13-foot horseshoe design section.

The field welding of the A 242 penstock steel in the shaft progressed very well with little rejection and chipping out of welds being required by the radiographing. A particularly rigid qualification test was used for the welders.

Only the top 20 feet of the 18-foot diameter and 389-foot long access shaft was concrete lined. The rock is excellent and there has been no incident of a falling rock. However, there is a natural tendency to think of such an occurrence being possible and for this reason, and for the reason that the costs of attaching all the services in the shaft to the irregular rock are high, it would have been better to have arbitrarily concrete lined such as access shaft. The power house walls will remain unlined and are satisfactory. There has been no incident of a falling rock in the first two years.

The thickening and raising of Balch Diversion Dam[3] has been completed. Temperature control measurements, strain gage measurements, and test drilling all indicate that the resulting dam is and should remain monolithic.

REFERENCES

1. Rockfill Dams: Salt Springs and Lower Bear River Concrete Face Dams. I. C. Steele and J. B. Cooke. Proceedings of the American Society of Civil Engineers, Paper No. 1737, Vol. 84, No. PO 4, August 1958.

2. Rockfill Dams: Wishon and Courtright Concrete Face Dams. J. Barry Cooke. Proceedings of the American Society of Civil Engineers, Paper No. 1746, Vol. 84, No. PO 4, August 1958.

3. Thickening and Raising the Balch Concrete Arch Dam. J. B. Cooke and J. E. Schumann. Sixth International Congress on Large Dams, New York, 1958.

UNDERGROUND POWER HOUSES IN ITALY AND OTHER COUNTRIES[a]

Closure by Claudio Marcello

CLAUDIO MARCELLO,[1] M. ASCE.—Mr. Lawton pointed out the main problem of the determination of the basic characteristics of an underground hydroelectric power plant.

The determination of the penstock slope for an underground hydroelectric plant is based on the well-balanced adjustment of various anthitetical elements. Besides the characteristics of the external ground profile and its geological conditions, these elements may be reduced to the following: (a) Length of the penstock and its cost; (b) length of the access gallery to the powerhouse and of the tailrace; and (c) characteristics of hydroelectric machinery regulation.

In the cases studied by the writer the tailrace and access gallery to the powerhouse as well as the electrical connection to the outside have generally represented a cost per linear meter of at least 10% higher than that of the penstock and therefore it would had been convenient to place the powerhouse as near as possible to the outside.

Such trend has been, however, in same cases partially limited by the higher cost of excavation of the tunnel for the penstock for slopes lower than 65% to 70%. For slopes higher than these values, excavation material is easily removable.

Moreover, the necessity of machinery regulation suggests that one should reduce the length of the penstock to remain within values that can assure limitation of the over-pressures in the penstock and effective machinery regulation.

Owing to this necessity, in power plants designed by the writer, the formula $\frac{L\,V}{g\,h}$ was made to equal about 0.80, where

L = the length of the penstock in m
V = velocity of the water in the penstock in m/sec
h = head in m between surge tank and discharge level in the tailrace
g = gravity acceleration in m/sec^2

a. Proc. Paper 1554, February, 1958, by Claudio Marcello.
1. Cons. Engr. and Technical Director, Societa Edison, Milan, Italy.

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THE SUDAGAI UNDERGROUND POWER PLANT, JAPAN[a]

Closure by Tatsuo Mizukoshi

TATSUO MIZUKOSHI.[1]—The writer wishes to thank Mr. Lawton for his discussion of the paper and for giving the writer an opportunity to supplement his paper.

The relative intensity of seismic forces at the surface and underground will be an important problem in the future. After completion of the Sudagai Project, the writer's company obtained the services of Dr. Okamoto who installed two seismographs in 1956, one on the underground generator floor EL 661.7 meters and the other on the surface ground EL 745.0 meters. These are accelerometers with the natural period of 0.1 second and 200 geometric magnifications and are provided with starters which operate in accordance with seismic tremor at the early stage. They recorded a medium scale earthquake on September 30, 1956, which had its epicenter about 180 kilometers southeast of the dam site. As a result of the investigation by means of response spectrum of this earthquake, a period of 0.4 seconds was dominant and the seismic force in the underground powerhouse was found to be 45 to 50 percent of that at the surface. Details of observations will be published in "geologie und Bauwesen" (1958) by Dr. Okamoto and the writer. The observations are still being made and more details will be published by Dr. Okamoto in due course.

With respect to the drainage tunnel built around the power plant, more details and an answer to the question are presented here as follows. It is because of inexperience in Japan of constructing an underground power plant situated immediately below the dam as the Sudagai Plant and of uncertainty of the quantity and pressure of leaking water through the rock, that a drainage tunnel, from which drain holes were drilled upward, was built around the powerhouse to check on the security against leakage. Drain holes drilled from the tunnel at the spacing of 10 meters are thought to be the checking holes as a test rather than a drainage curtain. If the leakage from the reservoir after storing the water is considerable, additional drain holes will be drilled for covering the powerhouse with a complete drainage curtain, or thorough grouting will be executed. However, when there is no increase of leakage through the drainage tunnel, additional drilling of holes or grouting of rock will not be performed.

In the Sudagai Project, this drainage technique was fortunately not useful, but as the site was located just below the dam and because of the complex nature of rock, 100 percent safety of the underground cave against the leakage from the reservoir could not be expected from geological surveys by

a. Proc. Paper 1555, February, 1958, by Tatsuo Mizukoshi.

1. Mgr. of Civ. Eng. Section, The Tokyo Electric Power Co., Inc., Tokyo, Japan.

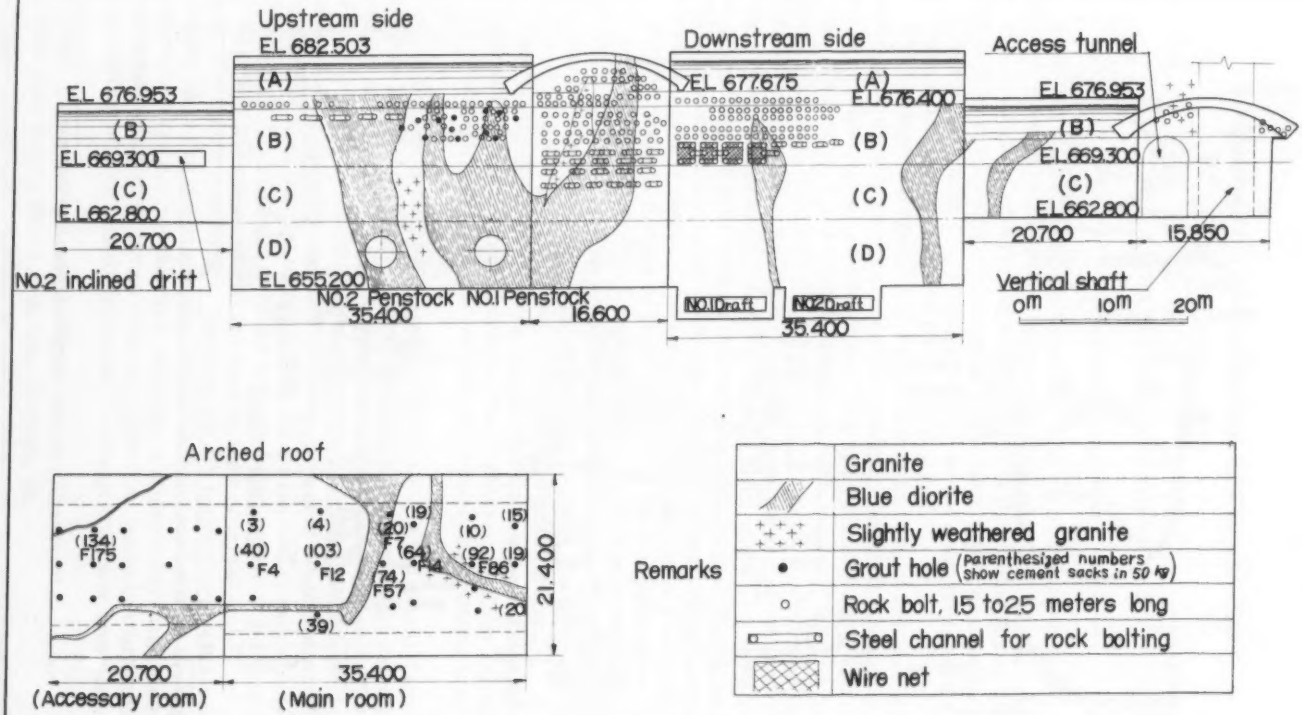
means of test borings and tunnel excavation nearby, and the construction cost of the drainage tunnel was not so high when compared with the total construction cost. For these reasons, the writer would adopt the same drainage practice if the Sudagai Project were planned again.

Grouting executed in the underground cave was for prevention of rock falling from the side walls in the course of excavation. Consolidation grouting for the geologically weak zone and contact grouting for filling in the interspace between the arch concrete placed and the rock behind were executed, but curtain grouting was not performed. Features of grouting and rock bolting are shown in Fig. 1. Rock bolting was performed mainly for prevention of rock falling during excavation and was not aimed at securing its effectiveness after placing concrete. Therefore, no consideration was given against corrosion of the rock bolts.

The modulus of elasticity of the foundation rock, measured by a jack in the examining adits at the dam site was 74,000 kg/cm^2 to 120,000 kg/cm^2 and Poisson's ratio of the rock core sampled was 0.195. The modulus of elasticity of test pieces made from rock samples taken underground at the powerhouse was 200,000 kg/cm^2 to 300,000 kg/cm^2. The tested pH value for the drainage water was 7.0.

The size of the air pipe installed through the partition wall between the surge chamber and tailrace tunnel is 0.96 meters in diameter and is contracted to 0.30 meters at the top, a valve being placed at the outlet of the pipe in order that the opening of the pipe may be suitably adjusted. According to the result of actual measurement of the behavior, the air hammering action was found to be minimized when the valve was fully open. Details of the air pipe are shown in Fig. 2.

Construction costs of power plants on the surface and underground depend largely upon local conditions of the sites such as geology and topography. As the construction cost of the underground power plant will be reduced by simplifying its structure, possibility for underground construction of hydro power plants will increase in the future. The writer's intention in the conclusion of the paper is as follows. Based on the experience gained through construction of Sudagai Power Plant underground, information on the leakage, seismic force, construction method, safety, and others, was or is now being available, so that the author affirmed that a simplified structure could have been possible with resultant reduction of construction cost if geological conditions had been more adequate.



GEOLOGICAL FEATURES OF UNDERGROND CAVE

Fig. 1.

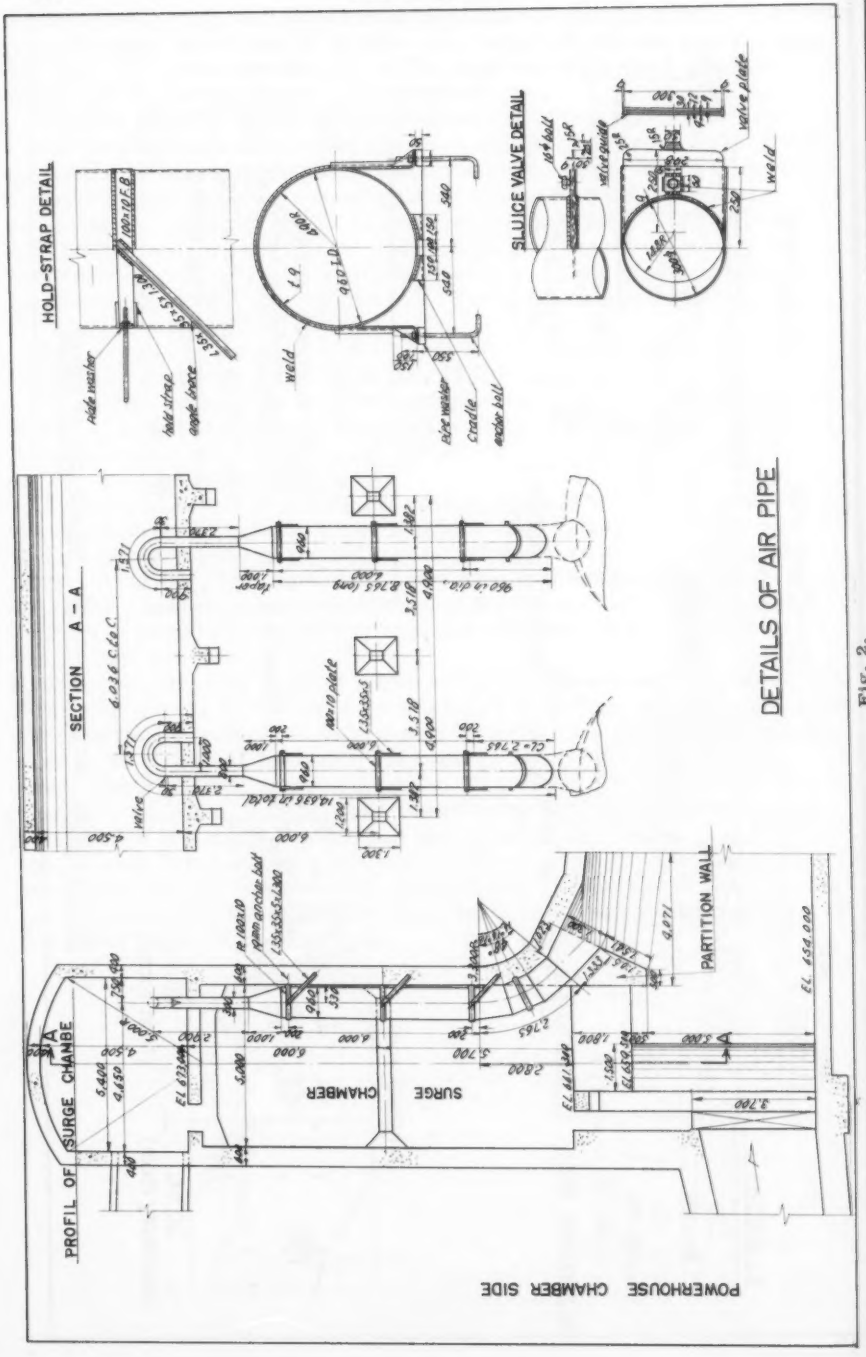

DETAILS OF AIR PIPE
HOLD-STRAP DETAIL
SLUICE VALVE DETAIL
SECTION A – A
PROFIL OF SURGE CHAMBER
SURGE CHAMBER
PARTITION WALL
POWERHOUSE CHAMBER SIDE

Fig. 2.

AMBUKLAO UNDERGROUND POWER STATION[a]

Closure by Andrew Eberhardt

ANDREW EBERHARDT,[1] M. ASCE.—Extensive discussions sent in by Messrs. Rodriguez and Nicolaou are appreciated.

Mr. Rodriguez' observations on cave-ins in the faulted areas and his description of additional short construction (or transit) tunnels dug by the contractor constitute a worthwhile addition to the description of the Ambuklao Project and complements well the original paper.

Mr. Rodriguez correctly points out that in an area of complex geology where rock conditions are difficult to predict, a vertical shaft access has one advantage over an access tunnel: the shaft transverses a small area of rock. Therefore, a few core holes will furnish more dependable geological information than a larger number of holes scattered along a long tunnel. Furthermore seepage should be less of a problem in a short shaft than in a long tunnel. There are, of course, other factors which have to be considered when planning the access to an underground installation. The most prominent factor (outside of the initial cost) being the convenience of access during construction. In this respect a tunnel is usually superior to a vertical shaft.

Mr. Nicolaou is correct in pointing out that the comparisons of the excavation volume made for the Guayabo plant and plants with vertical settings can not be directly applied to the design of the Ambuklao Project. This is, however, not only because of the presence of a valve chamber but also due to the peculiarities of the radial layout adopted at Ambuklao.

The results of Mr. Nicolaou's efforts to compare the excavation volume of four different underground plants (Table 1) deserves close scrutiny.

In the first place, some corrections are required regarding the Ambuklao data:

1. In the two vertical shaft power plants used in the comparison (no data on Binga Project is available to the author) there is a single valve provided directly ahead of each unit. At Ambuklao, however, for reasons independent of the horizontal shaft setting, each penstock is equipped with two valves in tandem. Fair comparison, therefore, requires that the ambuklao valve chamber volume (but not the volume of its access tunnel excavation) be reduced to what is required to house a single valve before this figure is added to the power chamber volume and used to draw conclusions.

2. The figure used by Mr. Nicolaou for the turbine vault excavation is apparently based on the assumption that all of the rock between the vaults was removed. This was not the case, although comparatively large amounts of overbreak occurred at the corners of the vaults. Since overbreak is primarily a function of rock quality (although the design will have some effect), the

a. Proc. Paper 1598, April, 1958, by Andrew Eberhardt.

1. Chf. Civ. Engr., Harza Eng. Co., Chicago, Ill.

excavation volume comparison should be based on the theoretical outlines rather than the actual overbreak lines. Furthermore, there is really no other choice because the figures of actual volume of excavation are seldom published. Accordingly the data used for other plants seem to represent the theoretical volume.

On this basis the excavation volume of the Ambuklao underground power chamber is as follows (dimensions used are those shown on the construction drawings):

Power station chamber	24,000 cu. yd.
Turbine vaults	3,400 cu. yd.
Valve chamber	4,700 cu. yd.
	32,100 cu. yd.

The figure used by Mr. Nicolaou is 21% higher.

3. The radial arrangement of the Ambuklao units results in increased length of the generator chamber but, on the other hand, it saves on the tail-race excavation downstream of the converging draft tubes. For this reason any reliable comparison of the excavation volume would have to be based on more than merely the power and valve chamber volume.

The most conscientious efforts, however, to arrive at a comparable figure for each project will be wasted if the various data collected is compared using an improper parameter. This, unfortunately, one finds to be the case when Table I prepared by Mr. Nicolaou is examined. The volumes of excavation are compared and the final conclusions are drawn on the basis of cubic yards of excavation per kw per unit.

It is rather easy to demonstrate that this particular parameter leads to entirely misleading results. Let us assume a hypothetical case of two underground power plants of the same head: one with a single 100,000 KW unit another with four 25,000 KW units. Let V_1 be the volume of the first chamber, and V_2 the volume of the second chamber.

If we assume that (as an example) the total volumes of both chambers are the same ($V_1 = V_2$) the amount of excavation per kilowatt of installed capacity in both plants will be the same too. On the basis, however, of "volume of excavation per kilowatt per unit" a four unit powerplant will appear much more attractive:

Single unit powerplant: $\dfrac{V_1}{100{,}000 \times 1}$ cu.yds/kw/unit

Four unit powerplant: $\dfrac{V_1}{100{,}000 \times 4}$ cu.yds/kw/unit

Similar results will be obtained for different assumed ratios of V_1:V_2 indicating that the introduction of the number of units in the parameter strongly favors plants with multiple units and distorts the comparison like a curved mirror. Even when the total volume of a four unit plant is assumed to be four times the volume of a single large unit plant, the cu. yd/kw/unit figure is the same for two plants indicating that the four unit plant is not more expensive to excavate than the chamber which was initially assumed to be four times less in volume!

These results unquestionably disqualify the parameter introduced by Mr. Nicolaou and invalidate his conclusions.

It may be of interest, however, to observe that smaller multiple units have

a certain edge over fewer and larger units when it comes to the excavation volume. This is because the output of a unit increases in proportion to the square of the runner diameter while the volume of the chamber housing the unit increases roughly as the cube of the runner diameter. On the basis of this imperfect assumption four 25,000 KW units would require only half as many cubic yards of excavation as a single 100,000 KW unit of the same head. But if the cu.yd/kw/unit yardstock is applied here too, the four 25,000 KW units will be made to appear as requiring only 1/8 of the excavation necessary for a single large unit.

It would seem that for plants of similar head a more reliable comparison can be made on the basis of cubic yards per kw of installed capacity. The number of units, however, in all plants should be brought up, it would seem, to the same figure in order to make up for the disadvantage the erection bay places on plants with fewer units. The total volume of excavation should be increased by the volume of unit bays so added.

If Mr. Nicolaou's data on Ambuklao is corrected and his table is revised as suggested above the results are as indicated in Table A.

TABLE A

	Total Volume Excavation Cu. Yd.	Volume of Excavation cu.yd/kw
Ambuklao	40,000**	0.40
Binga*	43,000	0.43
Isere Arc*	51,000	0.38
San Giustina*	49,000**	0.34

* Based on Mr. Nicolaou's data
** Volume increased by adding a fourth unit

It should be noted that the above comparison does not take into account the effect of specific speed on the size of the turbine. Ambuklao units have a conservative specific speed, therefore, they are somewhat larger than higher specific speed units would be.

Another possibility in the search for a reliable yardstick is the size of the equipment housed: the runner or generator diameters. This would permit direct comparison between plants regardless of the head and, therefore, it would open a wider range of projects for study.

The author is not inclined to agree that the final selection between the horizontal and vertical shaft setting will be necessarily influenced by geological condition. This would rather be an exception than a rule. The author would even venture to say that in most cases if the rock is found to be good enough to encourage going underground, it will be good enough for any type of shaft setting. The Ambuklao plant was built in rock which was quite poor in places and was never very good in general and was built without difficulties unusual to underground construction.

The excessive overbreak at the turbine vaults Mr. Nicolaou refers to cannot occur in a vertical shaft plant which has no such vaults and is, in some respects, stronger structurally. But excessive overbreak can occur in the vertical setting between the turbine pits. In either case, it still is a local overbreak and not something which would weigh heavily against any scheme.

On the other hand, the radial layout of the Ambuklao plant tends to aggravate these problems because of the tapering rock piers between the converging draft tubes. In this respect the horizontal setting with parallel units is preferable to the one with radial arrangement.

It is also difficult to see why the choice between the horizontal and vertical shaft arrangement should be affected by the "available construction equipment and construction techniques for underground excavation." The implication apparently is that the horizontal shaft setting introduces such excavating problems that it should not be considered unless the best equipment and construction methods are assured. In order to dispell these fears it will be sufficient to remind that the two recent horizontal setting power plants have been built in rather remote locations: one (Guayabo) in Central America, another (Ambuklao) in the Philippines. The third one is now nearing completion in India (Maithon). In all cases native labor was used extensively and there was no abundance of imported modern construction equipment or materials.

Mr. Nicolaou should be given credit for pointing out certain problems resulting from the horizontal shaft arrangement. The question of excavation, however, should not obscure in the mind of the design engineer other advantages (and disadvantages) of this setting. Particularly the permanent gain in the plant capacity due to increased efficiency of its hydraulic arrangement resulting from the elimination of one penstock bend and of the draft tube elbow. This feature has more significance in the plant arrangement selection than any other single factor.

UNDERGROUND POWER PLANTS IN CANADA[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The authors have provided an excellent treatment of some of the considerations peculiar to design and construction of underground power plants, and exemplified these by reference to the world's largest examples of this type—Kemano, Bersimis No. 1 and Chute-des-Passes, in Canada.

Their statement that "In any location it is essential that a thorough and comprehensive program of geological explorations be executed before a final layout is adopted" is most important in the design and construction of underground plants. Too often there is a tendency to skimp on geological investigation on the assumption that any additional work can be done during the course of construction.

The authors have noted the significance of powerhouse orientation with respect to the strike of foliations and bedding planes. They might well have drawn attention to the importance of orientation with respect to the strike and dip of joint systems.

The important consideration is that an underground powerhouse, or any other subterranean structure for that matter, is constructed from rock, usually heterogenous and normally non-isotropic. In adapting this material to the desired structural purposes the existing stress fields are materially changed and frequently an adequate material is damaged in the excavation process by the use of excessive explosives. Thus it is advantageous to observe whether any movement in the arch or walls of a major underground chamber occurs during excavation and later, as corrective measures can be instituted in good time without disturbing the construction schedule. Suitable proven instrumentation for such investigation is readily available.

The authors have referred to the importance of the layout of the hydraulic conduits adjacent to an underground powerhouse, in its bearing on the orientation of the chamber, tunnelling costs and hydraulic efficiency. This aspect merits full consideration in every case.

The trend in North American underground power developments is to arrange the penstocks so that the individual turbine branches are located at right angles to the longitudinal axis of the powerhouse. Thus, if the powerhouse is oriented so as to secure the maximum advantage from the geological formation—i.e. with longitudinal axis perpendicular to the strike of foliations and bedding planes—it is not so favorably located to achieve minimum tunnelling costs for the penstock branches. In Europe the penstocks are normally located at an acute angle to the longitudinal axis, so minimizing tunnelling

a. Proc. Paper 1670, June, 1958, by A. W. F. McQueen, C. N. Simpson and I. W. McCaig.

1. Chf. Engr., Power Dept., Aluminum Laboratories Ltd., Montreal, Canada.

difficulties due to geological conditions at the expense of somewhat more difficult penstock specials.

In conclusion a plea is made that engineers reporting to the profession on underground power developments should include in their treatment a realistic presentation of the significant aspects of the geological formations involved, preferably in the form of a block diagram correlated with the structures, and of the important rock physical properties.

BOX CANYON HYDROELECTRIC PROJECT[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The author has given an unusually fine and thorough account of design and construction problems encountered on the Box Canyon project. However, a few aspects warrant, it is believed, amplification which it is hoped Mr. Geuss will include in his closing discussion.

One such aspect is the asymmetrical shape adopted for the 90-foot high spillway piers in order to minimize unbalanced forces. Could the author indicate the magnitude of these forces? Were the air vents shown in Fig. 9 incorporated as a result of model test results?

The results achieved in the compaction of the sand in the spillway area by vibroflotation are interesting. The paper implies, but does not state explicitly this was done under water. Would the author care to clarify this? The data given on the improvement in the relative density are useful. Does the author consider a closer pattern for insertion of the vibroflotation unit would have achieved a material additional improvement?

The author has noted, in respect of the spillway weir, that "Consolidation and curtain grouting was carried out in both abutments, in addition to grouting of the sand foundation where the steel sheet piling terminated against the rock abutment faces." Would the author care to describe the method used for the latter, i.e., grouting of the sand foundation at the sheet piling termination?

The use of the horizontal arch under the spillway is interesting. It brings to mind a somewhat analogous case in Norway where a dam was constructed on a horizontal arch spanning a rather deep filled gorge. The author notes an allowance of 0.1 gravity was made for earthquake acceleration. Was this applied in the vertical as well as the horizontal directions?

The author has noted the effect of the spillway upstream apron in deflecting the flow from the main river channel into the forebay channel. Could the author state the actual head loss between the river just upstream of the spillway and the unit intakes? Is there any marked irregularity in flow distribution?

a. Proc. Paper 1672, June, 1958, by Arthur P. Geuss.

1. Chf. Engr., Power Dept., Aluminum Laboratories Ltd., Montreal, Canada.

UNDERGROUND POWER PLANTS IN SCOTLAND[a]

Discussions by C. F. Colebrook, John B. Milne, J. Guthrie Brown and F. L. Lawton

C. F. COLEBROOK.[1]—To be able to place the power house underground in almost any position has considerable merit in that full advantage can be taken of the topographical and geological conditions and in areas liable to severe weather conditions during the winter months it is possible to continue with construction work without interruption, thus saving time and reducing overhead costs.

It is noted that the difference in the estimated cost of the alternative underground schemes 2 and 3 for the Glenmoriston works was relatively small and it would be interesting to know how much cheaper than scheme 1 are the estimated costs of both schemes. A surface power house would appear to be cheaper than an underground station, especially when one has to make allowance for possible grouting and bolting operations such as those carried out at Glenmoriston, and it is therefore surprising to learn that scheme 3 was estimated to be cheaper than scheme 1.

It would be interesting to know to what extent maintenance costs would be reduced in scheme 2 and a fuller explanation of the advantage of underground stations in respect of personnel requirements and the siting of transformers would be appreciated.

It is noted that Ceannacroc was originally planned as a surface station but it was finally located underground. Since a tail development was adopted in this case is it to be understood that the increased cost of an underground solution was more than compensated for in time and cost by the installation of a machine similar to those ordered for the Glenmoriston station.

Referring to the dams it is noted that Trief cement was extensively used and it would be interesting to know what was the estimated saving in cost. Would the author express an opinion as to the probable minimum size of dam for the economic use of Trief cement? The use of pre-cast facing slabs of special design is of interest in view of the difficulty experienced in obtaining joiners. Apart from the question of cost and convenience pre-cast work has a very fine appearance and a dam which was recently completed in granite country in Northern Ireland for the Belfast Water Commission with precast concrete facing blocks has a most attractive appearance which compares favorably with the fine granite masonry of the Silent Valley dam in the lower reaches of the valley. Both dams enhance the natural beauty of the wild mountainous country in which they are located.

It is noted that although three methods were used to deal with sections of poor rock in the tailrace tunnel to the mouth of the River Moriston it is

a. Proc. Paper 1675, June, 1958, by C. M. Roberts.

1. Sr. Engr., Binnie, Deacon & Gourley, Westminster, England.

surprising that it was not necessary to completely line sections of this tunnel and that sealing "small" rock with sprayed concrete using the "Aliva" gun was sufficient to stabilise the rock sufficiently to resist scour by the discharging water.

Would the author state more fully the basis for the design of the roof in the Ceannacroc station? The roof was designed to carry a load equivalent to a depth of rock of about 1/3 span. Was the same basis adopted for the design of the Glenmoriston station where the rock was much more shattered? It would be interesting to know whether the roof of the power house which was concreted prior to the removal of the dumpling suffered any cracking or other damage as a result of the subsequent blasting.

The initial 1350 ft. length of Glenmoriston tailrace tunnel has a gradient of 1 in 50 which results in submergence of the remaining length of tunnel. It would appear that on a sudden acceptance of load any air in the roof of the inclined section of tunnel would be rapidly expelled with this arrangement and it would be interesting to know whether there would have been any hydraulic advantage in shortening the length of tunnel subject to partial emptying by steepening the gradient still further. This might however lead to increased construction difficulties, should the gradient exceed about 1 in 15 since winch haulage may then become necessary, which would not be offset by any hydraulic advantage.

Referring to the considerable difficulties experienced in dealing with weak rock and water in the Glenmoriston station, whilst it was possible to employ the technique of rock bolting etc. it would, of course, have been preferable to avoid altogether the necessity of building an underground station in such difficult ground. Did the various tunnels which were driven towards the power house indicate much sounder rock within a short distance of the position originally selected for the power house, in which case might it not have been advantageous to resite the house in better ground? The problems involved in site investigation for underground power houses are very difficult since even an extensive program of borings will not necessarily reveal a large fault, nor does it give much information on the degree of fragmentation of the rock. Borings are however essential and the writer would be glad if the author would enlarge on this subject for the guidance of those who have not the author's experience.

If it is not possible to select a position free from water troubles the writer has no objection to the provision of a concrete arch for support of the roof and a false, lightweight, waterproof roof, of aluminum or concrete, below bolted to the main concrete roof for its support.

The author is to be congratulated on the completion of a most interesting scheme for the North of Scotland Hydro-Electric Board involving the construction of two underground power stations, the first of their type in the United Kingdom.

JOHN B. MILNE.[1]—The entirely different approach of the two civil engineering contractors in excavating essentially similar chambers is noteworthy.

The method adopted at Ceannacroc has been described.

At Glenmoriston, the access tunnel drive divided at the machine chamber.

1. Resident Engr., Lower Moriston Works, North of Scotland Hydroelectric Bd., Dundreggan, Glenmoriston, Inverness-shire, Scotland.

One drive progressed across the off-loading bay, turned and ramped up along the line of the upstream wall to a level just below the arch springing at the north wall. From there two faces were opened and the roof arch excavated.

The other drive was pushed along the line of the downstream wall at main floor level. This drive extended past the north wall to the lift shaft and the shaft stope was started. Also from this drive tunnels were driven inwards and downwards to the draft tube pits and high pressure tunnels. Excavation was thus going on at several levels at once. The "dumpling" was removed following concreting and cavity grouting of the roof arch.

Concrete Arch

The decision to concrete the roof arch was taken as soon as the results of the geological exploration were known. Concrete was placed by a 6" concrete pump sited at the foot of the Intake shaft. The work was shuttered in sections about 15' long, the shutter being mounted on a Bailey bridge gantry which was, in turn, mounted on two rail bogies. Following completion of concreting, this mobile gantry, illustrated in Fig. 1, was later used as a working platform for the initial cavity grouting.

Grouting and Lining

The author refers briefly to the additional support to the walls and it may be of interest to follow the steps which led up to the methods which were adopted.

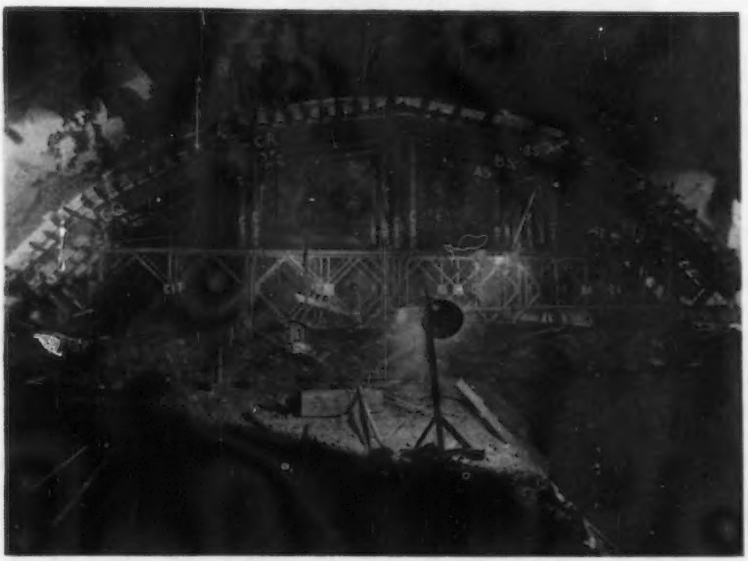


Fig. 1.

Just before work started, for reasons of economy, the width of the machine chamber was reduced by 2' -0". This decision was reluctantly taken as it precluded the use of the original contract design which envisaged a concrete lined hall with inner brick walls forming a large cavity between brick and concrete. This design allowed easy access to the side drains which was essential for clearing away the "rock salts" which are heavily deposited in underground excavations in the area.

As the excavation proceeded it became obvious that support for the rock walls was necessary, the problem being complicated by the smaller space now available. The rock was lying at about 10 degrees to the vertical in beds roughly 2' to 4' thick. Rock bolting was decided upon and a bolt length of 9' -0" was chosen as adequate to "stitch" the slabs together.

Although there were several points of leakage in the hall excavation the true nature of the rock really became obvious when drilling for the rock bolts started. Each hole produced a small seep or trickle of water.

This knowledge that the rock was water-bearing to a high degree made grouting very necessary and as grouting was not practicable without first sealing the rock face, the walls were concrete lined.

The process of grouting was quite standard except that ways to increase

Fig. 2.

the travel of the grout were sought. One or two types of wetting agents were tried and were successful in increasing the distance travelled by the grout, by 20% to 30%. Also successful was the use of colloidal grout; as this was a more straightforward process, it was used for the tertiary grouting.

The main purpose of the grouting was to drive the water table upwards above roof level and from north to south to the end of the short length of access tunnel lining as well as to the chosen relief area at the south end of the machine hall. As already stated in the paper, relief holes were drilled from the surge shaft up and over the roof of the hall. These holes were successful in draining 900 gallons/hour into the tailrace tunnel: most of the remaining water. The roof has no false ceiling.

Prior to grouting in the machine chamber, some trial grouting was done from the surface. Six holes were drilled from the cut-off trench under the east wall of the intake—this was close to the north end of the machine hall—see Fig. 6. These holes were stage grouted at pressures varying from 40 lbs./sq.in. at 50' depth to 200 lbs./sq.in. (gauge at surface) at 300' depth. The holes were drilled using hand held machines and 16' extensions. Bits started at 42 mm. diameter and reduced to 38 mm. The results of this grouting were disappointing, only small amounts of grout being taken, probably due to the fact that the holes were inclined so closely to the lie of the rock and also to the fact that, as stated in the paper, the rock fissures were clay filled. Following this trail it was decided to go for fissure grouting entirely from inside the machine chamber.

Fig. 2 shows the final appearance of the machine hall.

J. GUTHRIE BROWN.[1]—There is much of interest in this paper, which gives a good indication of present-day trends in design of medium-sized developments in Great Britain.

Confirming the comments solely to the underground works, it would be interesting to know the reason for the difference in treatment of the arch ring in the underground power stations at Ceannacroc and Glenmoriston, respectively. In the former case, the ring sits right on the edge of the vertical rock face with the possibility of future spalling of the rock. This is emphasised by the photograph, Fig. 12, which shows the irregular finish to the upper part of the vertical face. At Glenmoriston, however, the ring is set some 4 feet back from the vertical face which seems more satisfactory. It is noted that in this station the vertical rock walls had to be strengthened by rock bolts with a 12" thick concrete wall, the whole being pressure grouted and relief holes provided as a further precaution against a build up of water pressure. A brick wall lining with an air space behind into which leakage water from the rock could drain and be led away has been found to be very satisfactory in such cases. It eliminates the necessity for the thick concrete wall and provides a much drier station.

In this connection, the exposed rock walls in Ceannacroc are held to be attractive by architects, but they cannot possibly appeal to the maintenance staff. It is impossible to guarantee that leakage or excessive dampness will not occur in the future such as to be detrimental to the machines. Cases have arisen where the conditions have deteriorated to such an extent as to necessitate guniting the rock face at a later date, the result being a rough

1. Sir Alexander Gibb & Partners, Cons. Engrs., London, England.

concrete wall which one should imagine could be anathema to the architects.

Opinions differ widely on the wisdom of having inlet valves in the same underground chamber as the machines, in view of the possibility of a burst. Certainly in high head stations it has serious potential dangers, but in the low head station described in the paper the risk appears to be acceptable.

In the case of the smaller underground stations, there must be a minimum size of installed plant below which it is uneconomical to adopt an underground station. The author quotes 505 cu.yds/MW as the generating station space in relation to plant installed at Ceannacroc and 329 cu.yds./MW for Glenmoriston. At Kariba, in Southern Rhodesia, with an underground station having an installed plant totalling 600 MW the ratio is 220 cu.yds/MW, which shows a marked improvement over the above two smaller stations.

F. L. LAWTON,[1] M. ASCE.—The paper is a very interesting one, dealing as it does with Ceannacroc and Glenmoriston, the first two main power stations in Scotland built completely underground. The paper is also noteworthy in that it indicates the underground solution was the economic one for two relatively small plants, i.e. 16 and 32 MW.

In referring to the Loyne and Cluanie dams, the author noted the use of precast facing slabs, of ordinary Portland cement concrete, eliminating the use of formwork, proved to require much less labor and was responsible for a considerable increase in progress. Would the author care to state the magnitude of these economies? Was the finished work as attractive? Is it standing up to Scottish weather? Does the author consider the slabs are so well incorporated in or bonded to the mass concrete that a pleasing surface appearance will exist over forty or fifty years?

The author has described the rather interesting provisions permitting salmon to reach Ceannacroc, where they are relieved of their spawn which is taken care of in a nearby hatchery. Could the author state the cost of these provisions relative to the total cost of civil engineering works?

Although the paper does not state clearly the manner in which the grouting of the Glenmoriston machine chamber was carried out, it is assumed the holes for the wall grouting were horizontal. Would the author confirm this? It would appear from the paper that the three-stage grouting was carried out to secure water tightness, in addition to improvement in stability, in the "weak and shattered" rock walls which were rock bolted with 9 ft. long bolts on 4 ft. centres both way. The paper would be of materially greater value if the author could amplify his brief remarks on the geology of Glenmoriston station by a block diagram showing the principal weaknesses in the geological structure and the relationship thereto of the remedial measures undertaken.

How did the unit cost of excavation and remedial treatment—that is, rock bolting and grouting—at Glenmoriston compare with that at Ceannacroc?

Could the author state the time-volume of ventilating-air movement through the two stations?

The author's concluding observations on the importance of geological exploration and on the properties and behaviour of rock at depth cannot be too strongly endorsed.

1. Chf. Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

ROCKFILL DAMS: PERFORMANCE AND MAINTENANCE[a] OF DIX RIVER DAM

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—Reference is made in the paper to the two shots in the east cliff which each resulted in about 50,000 cubic yards of rock within the confines of the dam. Would the author care to indicate whether these mass movements of rock, although only a small part of the total volume of the dam, have contributed to leakage at the surfaces on which the movements were deposited?

The author's deductions from the settlement records are extremely significant, especially that indicating settlement and downstream movement may well continue over a period of some 50 years or more. This demonstrates the merit of constructing a high rockfill dam with a moderate convexity upstream.

This continuing settlement and downstream movement can only occur through the crushing of point contacts in the coarser rockfill and a slow consolidation. It would be of interest to know the physical properties of the rock and if there has been any deterioration since the dam was built.

The experience at Dix dam demonstrates clearly the difficulty of securing a trouble-free concrete facing on a high rockfill dam.

It would add materially to the value of the paper if the author could indicate the total maintenance costs for the Dix rockfill dam since it was built.

The author is to be congratulated on a noteworthy contribution to engineering knowledge on rockfill dams, more particularly since the paper covers some 33 years of observation following construction.

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a. Proc. Paper 1683, June, 1958, by Lewis A. Schmidt, Jr.

1. Chf. Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

ROCKFILL DAMS: COGSWELL AND SAN GABRIEL DAMS

Discussions by W. A. Brown, John V. Spielman, P. L. Aitken, F. L. Lawton, Claude A. Fetzer and Elton G. Knight, and Harris C. Porter

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W. A. BROWN,[1] A.M. ASCE.—Mr. Baumann has seriously questioned the commonly accepted belief that large dumped rockfill dams are highly pervious. Unfortunately, a physical demonstration of impermeability within the body of a high rockfill dam is lacking and there will be much reluctance by engineers to accept this premise without further proof.

The leakage at Cogswell Dam was reported to have reached a maximum of 130 c.f.s. through the timber facing. It would ordinarily be presumed that this leakage flowed through the interstices of the rock at the base of the dam. Mr. Baumann's analysis would suggest a relatively high zone of saturation within the center of the dam and accompanying pore pressures while such leakage was occurring. It follows that the downstream slope stability would have been reduced.

As higher rockfill dams are constructed, prudent design should dictate an investigation of the degree of self-compaction in existing high rockfill dams and the resulting permeability. Such investigations might include borings and piezometer installations in existing structures, plus largescale laboratory compression tests with correspondingly large loadings on quarry rock samples.

San Gabriel Dam No. 1 has a large central core of low permeability. Because of the flat core slopes, the dumped rockfill shells serve as weighting zones. In this case, consolidation of the shells has little or no effect on the stability of the dam. Even if self-compaction reduced the permeability of the rockfill shells, they would unquestionably be much more pervious than the core. In the writer's opinion, this design is as nearly unquestionably safe as any dam can be.

It is desired to compare this design to one wherein the central core width has been reduced to a maximum of, say 10 or 20 feet, at the crest and having side slopes of 0.2 horizontally to 1.0 vertically, or steeper.

Transition zones between the core and rockfill shells immediately assume great importance to prevent migration of the fines and piping into the dumped rockfill shells.

Vertical post-construction settlement of the rockfill shells may appear to be of no more importance than in the case of the San Gabriel No. 1 type of design and of much less importance than in the Cogswell design. The formation of longitudinal cracks on the crest of such dams has frequently been observed. It is ascribed to differential settlement of the rockfill relative to the

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a. Proc. Paper 1687, June, 1958, by Paul Baumann.

1. Principal Engr. of Dams, California Dept. of Water Resources, Sacramento, Calif.

core and a slight separation at the interface. Such cracks are generally considered harmless. They do, however, indicate local lack of lateral support for the core.

The thin central core, by itself, has little stability. It is essentially a membrane which is solely dependent upon the passive pressure of the downstream rock shell for stability. Conventional design methods recognize that this passive pressure must be sufficient to resist the reservoir pressure and the active pressure of the upstream rockfill shell. A sliding wedge analysis indicates that the thinner the central core is, the steeper the downstream rockfill shell slope may be. If a high value of coefficient of internal friction for the rockfill is adopted, it can even be shown that the downstream slope may be as steep as the angle of repose of the rockfill, neglecting allowances for seismic loads. This design approach should not be taken, however, without consideration of deformation.

Mr. Baumann has shown that a dumped rockfill achieves consolidation through its own weight. Crushing of the smaller rock particles and compaction of the material in the rock voids takes place due to weight. An equal degree of intimate contact and crushing is not developed in a horizontal direction. Yet with the vertical core, the water load is applied horizontally instead of toward the foundation. It is reasonable to expect, therefore, that the water load deflection will be larger in proportion to the applied load than in any other type of rockfill dam. Further, lateral deflection of an insufficiently large rockfill shell due to horizontal water load can cause failure of the core by rupturing long before a shear failure will occur in the rockfill shell.

The records of a few of the older dams in California, which had vertical concrete core walls and dumped rockfill shells, verify the above expectations. One in particular, having a maximum height of about 140 feet, deflected slowly downstream over a period of years, a maximum of nearly 13 feet at the crest. The movement was finally arrested by dumping a large volume of additional rockfill on the downstream shell to increase its mass. The shell slope was on the angle of repose of the rockfill. Fortunately, the reinforced core wall was able to undergo the deflection without complete failure in this instance.

The most adverse characteristic of a loose rockfill is its capability to deform non-elastically—from its own weight or from applied load. Post construction consolidation can be minimized by appropriate placing methods and sluicing, but neither the magnitude of this nor that of water load deformation can be pre-determined with any mathematical certainty. Dams of the Cogswell type may be expected to approach a limit of deformation within a reasonable time, and failure to provide a design which will accommodate that maximum deformation usually only means excessive leakage and repairs to the upstream facing. It is suggested, however, that much more serious consequences may be the result with the central core type of rockfill dam if too much reliance is placed upon the downstream shell as a structural supporting member rather than as a weighting zone as in the case of San Gabriel Dam No. 1.

This only emphasizes Mr. Baumann's comment that the design of a loose rockfill dam by far exceeds in simplicity the analysis of its inner stability. His conclusion that a compacted fill dam should be given preference where possible over a loose rockfill dam is fully justified on the basis of the experience on these two dams.

JOHN V. SPIELMAN,[1] A.M. ASCE.—San Gabriel Dam No. 2 (now Cogswell Dam), in its brief history, caused considerable concern to those responsibly connected with the project. Mr. Baumann's paper is of value because it describes the features of design, construction, performance, and repairs of a dam in which valuable lessons may be learned from the mistakes in design and construction, as well as in the corrections that were made.

The writer, as a design engineer with the Los Angeles County Flood Control District from 1932 to 1942, feels qualified to comment on some of the features of design and the performance record.

San Gabriel Dam No. 2 was unusual in many respects. First, in design and specifications for construction, the current practices of the time were not followed. Mr. M. M. O'Shaughnessy once described to the writer the two principal methods by which successful rockfill dams had been built. The method he preferred and used at Morena Dam (Transactions, Volume LXXV) is to put only large pieces of clean sound rock into the fill. Sluicing is not required in this method because large stones bear directly on large stones and little settlement is likely (except perhaps in very high dams). The second method is to put most or all of the quarried material into the fill, meanwhile sluicing it so thoroughly that the settlement occurs immediately during the construction period.

At San Gabriel Dam No. 2, an unfortunate composite of the two methods was followed. Sluicing was not provided for in the specifications. An attempt was made to keep out most of the quarry fines, but it was found that the grinding action of the equipment, particularly the cat treads, on the surface of each of the 25-foot high lifts, resulted in 8" to 12" of ground-up rock. If this had been sluiced down into the voids before beginning the next lift, it is believed that the principal cause for subsequent settlement would have been eliminated; but the surface was only scarified before beginning the next lift. Consequently, settlement came when the fill became wet from rainfall.

The author states the amount of settlement as a percentage of the height of fill. It may seem more impressive to express it in actual feet of accumulated settlement. In the storm of January 1, 1934, the crest at station 4 + 80 settled 5.8 feet in one night and for the six-month period from the time of first completion of the crest, about December 1, 1933, to June 13, 1934, the aggregate settlement at this point was 13.6 feet. Subsequently, the applied sluicing caused an additional drop of 3.9 feet, making a total of 17.5 feet by August 8, 1935.

The freeboard was maintained during this period by adding rock to the crest from time to time.

The original design freeboard was 20 feet. The 1938 storm filled the reservoir and produced a head of 11.4 feet on the spillway. Therefore, it might be concluded that if the 1938 storm had occurred in January of 1934, the probable settlement combined with the high reservoir level would have resulted in a disastrous overtopping of the dam.

The unusual and supposedly "more flexible" design of the laminated concrete facing with the "floater slab" connection to the cutoff never had a "fair chance" because the conditions for which it was designed were so greatly exceeded. Perhaps with a nominal amount of settlement as is experienced in most rockfill dams, this facing would have been satisfactory. It is the

1. Supervising Engr., Supervision of Dams for Safety, State Dept. of Water Resources, Los Angeles, Calif.

writer's conclusion, however, that there is no justification for more than a simple heavy reinforced-concrete slab.

The temporary timber facing applied to San Gabriel Dam No. 2 was quite successful. There was sufficient seepage so as to aid the settlement during the period of water loading and the life of 12 years was long enough for settlement to practically cease.

Finally, in 1947, a permanent reinforced concrete facing of simple, rugged design was placed on the dam.

Many existing rockfill dams have had sufficient settlement to result in serious cracking of the facing so that water loss is of consequence. Thus, San Gabriel No. 2 Dam is now, perhaps, a tighter and better dam than many that did not have the exaggerated troubles that this dam experienced.

The writer notes that San Gabriel Dam No. 1 is now labeled a rockfill dam, but on checking, finds that, despite the large central portion of compacted earthfill, over 50 per cent of the composite volume is loose rock fill; thus it is a rockfill dam according to definition. However, if the outer slopes, which are 1 on 3 both upstream and downstream, were as steep as some more recently designed dams that is, about 1 on 2.25, then the compacted earth fill would comprise over 50 per cent of the total, and it would then be considered an earth dam.

The small amount of settlement, 0.4 foot, emphasizes the vast difference between this dam and the usual rockfill dam.

P. L. AITKEN.[1]—The Author is to be congratulated in drawing attention to so many practical points to be considered in the design and construction of underground power stations.

In the case of the lower power station at Glenmoriston the estimates for a conventional and underground development showed only a marginal difference. The underground head development could only be justified if the rock through which the tailrace tunnel was driven was good enough to be left substantially unlined. This proved to be the case so that the selection has been completely justified, notwithstanding the poorer rock encountered at the power station itself.

An incidental advantage in the decision to adopt an unlined tunnel was that it gave the contractor the opportunity to use pneumatic wheeled tractors for mucking in the tunnel rather than narrow gauge rail track and skips, which is the usual practice in the smaller lined tunnels in the North of Scotland. The contractor developed a special tipping body for the pneumatic wheeled tractors which delivered the muck from the face to hoppers at the adits from which conveyer belts took it to tips.

The balance of advantage was finally more than turned in favor of the underground station because it eliminated the need for four miles of steel tower transmission line down the narrow glen. Incidentally, it also represented the last of many changes in the interests of amenity made to the original scheme proposed in 1936. The most important change had been made earlier when the North of Scotland Hydro-Electric Board in promoting their post-war scheme decided to limit the reservoir at Dundreggan to a mere headpond and thus save the flooding of agricultural land which would have resulted in some depopulation of the glen. One result of this sacrifice of storage is that

1. Chief Hydraulic Engineer, North of Scotland Hydro-Electric Board, Edinburgh, Scotland.

control of the run-off from the lower catchment area is lost and makes the operation of the stations more inflexible than it would otherwise have been.

In addition to paying attention to amenity both in respect of the scenery of the glen and of its economy, the North of Scotland Hydro-Electric Board is legally bound to avoid as far as possible injury to the stock of salmon. The author has reviewed the arrangements for the protection of fish-life including the establishment of a hatchery for the eggs of salmon trapped above the tailrace of the upper power station at Ceannacroc. Only a fraction of the fish ascending the River Moriston reach there, however, as there are five miles of good spawning gravel in the river immediately below the trap. So that full use could be made of this spawning gravel, it was agreed when the scheme was promoted that a minimum flow of 186 cusecs would be guaranteed in the river below Ceannacroc Power Station. This flow is 38% of the average flow for power at Ceannacroc Power Station and compares with an estimated dry weather flow of about 27 cusecs in the stretch of the river in question. The steady release of such a flow when the main machine in the power station is shutdown necessitated the installation there of the 4 MW machine.

Another point of interest in connection with fish is that it was decided to consider the five miles of river between the lower dam at Dundreggan and the point of discharge of the tailrace as a fish pass and to prohibit angling, the rights to which the North of Scotland Hydro-Electric Board purchased. So as to make this stretch of river suitable for the passage and refuge of fish at the lower range of compensation flows, improvements were made to the river bed and channel. Considerable success was achieved at low cost by using stone-filled gabions to form short piers running from the river bank at right-angles to the flow in order to concentrate the river flow into a narrower channel at places where it would have been too shallow for fish movements.

It is intended to take advantage of the experience gained on the Moriston and other underground stations when carrying out two new schemes which the Board have in hand. One of these involves a 24,000 kW power station and the other is a 400,000 kW pumped storage power station. The latter station would be 1,100 feet below the reservoir in what has been proved by drilling to be good sound granite.

F. L. LAWTON,[1] M. ASCE.—The author's paper on the 280 feet high Cogswell and the 380 feet San Gabriel dams is an unusually valuable addition to available information on rockfill dams.

The author, in referring to the cutoff wall between elevs. 2235 and 2320 of the Cogswell right abutment, noted that the placement of rockfill in the V-shaped gap from a greater height than the specified maximum of 25 feet made arching between the fill slope and the abutment wall unavoidable. Is there positive proof of this, in view of the later observation on the settlement in the previously-dry rockfill occasioned by the storm of December 31 - January 1, 1934?

The author's statement that the initial settlement of some 4 percent at the right abutment, due to the storm, was only increased to 6 percent by several months of pumping clear water, through the facing, into the rockfill is most important, particularly when contrasted with the 6 percent average

1. Chf. Engr., Power Dept., Aluminium Laboratories Ltd., Montreal, Canada.

settlement secured at San Gabriel dam by the use of about twice as much volume of sluicing water as rock. It can, it would appear, be concluded that ample sluicing water is imperative, a condition generally confirmed by practice on most rockfill dams.

The shrinkage of rockfill after placing in Cogswell dam, during construction, as shown by Fig. 11, at approximately 18 percent must be attributed to consolidation of dry rockfill placed in lifts not exceeding 25 feet. Does the author consider the shrinkage during construction is materially changed by dry deposition in higher lifts, say 50 to 75 feet?

The deduction by the author that "--rockfill dams of major height can by no means be considered immune to pore pressure, especially in the interior of the lower part of the dam where densities would normally be greatest" is significant because of the probable formation of "a core of low permeability" in the lower portion. This appears to be confirmed by the mechanically compacted material in zone 3 of the San Gabriel dam, resulting in a percolation rate of 3 feet per year under a gradient of 1 to 1. It is also confirmed, it would appear, qualitatively by the 3 inch core from the San Gabriel dam.

The observation that the San Gabriel dam "Under the maximum water load deflected downstream 0.11 feet at the crest and returned (elastically) within a fraction of an inch to its position before the flood when the reservoir was emptied," is a confirmation of a little known property of a rockfill dam.

The experience with the upstream facing on the Gogswell dam is one more case of the difficulty of securing trouble-free performance with any rigid or semi-rigid facing.

In view of the different design of the two dams operated by the one owner, it would add to the value of the author's very fine paper if the maintenance costs for the Cogswell and San Gabriel dams over the same period of years could be stated.

CLAUDE A. FETZER,[1] A.M. ASCE and ELTON G. KNIGHT.[2]—The factual data presented on the performance of Cogswell and San Gabriel Dams are a major contribution in the advancement of the design of rockfill and earthfill dams. These data are particularly important because both dams have been subjected to spillway floods. The settlement and movement of the upstream face of Cogswell Dam (San Gabriel Dam No. 2) during the floods and the type and quality of rock obtained from the quarry for San Gabriel Dam No. 1 should be indelibly impressed on the minds of all dam designers.

With reference to the settlement and permeability of rockfill dams, it is conceivable that rockfill dams are not a mass of large rock with larger interstices; however, it is doubted that the voids at the base of rockfill dams are filled with high density material. The conclusions that the filler material has a density was based upon a computed density of 132 lb per cu ft for the material in the voids of San Gabriel Sam No. 2. The computed density was based upon the following assumptions:

1. Average volume per truck amounted to 11.5 cubic yards.
2. Loose rock on trucks had 40 percent voids.
3. Quarry dust equalled the amount permitted by the specifications - 3 percent of rock weight.

1. Chf., Soils Design Section, U. S. Army Engr. Dist., Los Angeles, Calif.
2. Chf., Investigation Section, U. S. Army Engr. Dist., Los Angeles, Calif.

4. Seventy-five percent of rock remained uncrushed and 25 percent of the rock combined with quarry dust to fill the rock voids.

5. Specific consolidation at the base of the dam was equal to twice the average consolidation.

No attempt will be made to evaluate these assumptions; however, the computed density of the filler material is questioned. A density of 132 lb per cu ft is a very high density, and it is much higher than the densities obtained in test fills built for San Gabriel Dam No. 1(1). In these test fills, quarry material passing a 6-inch by 9-inch "grizzly" opening was placed in 25-foot lifts and throughly sluiced at the time of placing. Densities were obtained in the fill by sinking shafts and testing the materials at 5-foot intervals. The average dry density of the fines was found to be 89 lb per cu ft, and the dry density of the rock and fines averaged 122 lb per cu ft. In comparison the densities obtained on the same material that had been placed in thin layers and rolled with a heavy roller averaged 122.2 lb per cu ft for fines and 144 lb per cu ft for rock and fines.

In the test fills built for San Gabriel Dam No. 1 the entire weight of the fill was carried by the mixture of rock and fines; whereas, in a rockfill dam it is presumed that the weight of the fill is carried to a considerable extent by point to point contact of the uncrushed rock. If this is true, the compaction of the filler material in a rockfill dam would probably be less than that obtained in a sluiced fill of fines and small rock, and would undoubtedly be less than that obtained in a rolled fill.

In conclusion it would appear that more definite information on the density of filler materials at the base of rockfill dams is needed prior to assuming that the voids are filled with high density material.

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1. Bauman, Paul, "Design and Construction of San Gabriel Dam No. 1," Transaction, ASCE, Vol. 107 paper No. 2168, 1942.

HARRIS C. PORTER,[1] A.M. ASCE.—The author has made a thorough summary of the features of special interest concerning the design, construction, and performance of the two rockfill dams treated in this paper. Very little remains to be added in the opinion of the writer who is in general agreement with the author's conclusions and wishes however to offer supplemental information concerning details of construction of Cogswell Dam which may be of interest to those who encounter situations of a similar nature. For that purpose this discussion is limited to rockfill, abutment preparation and cut-off and floater slab construction.

The quarry developed in Devils Canyon provided rock of excellent quality in adequate quantity to maintain the construction schedule of completion of dam fill and facing prior to the winter of 1933-34. The grading of rock to meet specification was assured by careful sorting in the quarry. Normal dumping segregation was experienced in the fill. Careful removal by power equipment of the surface accumulation of quarry chips, dust, and temporary construction road topping prior to the construction of the next lift appeared

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to be adequate. However, the sluicing effect of rain storms very clearly showed otherwise. The immediate settlement observed when December storms wet down the dry dumped fill emphasized most drastically the importance of sluicing rock fill during construction. Also it has been noted that settlement varied greatly in adjacent portions of the fill. It is the belief of the writer the variation in settlement was due in part to the irregular slope of the south abutment as may be seen in Fig. 2 and Fig. 3(b) and in part to construction methods used to avoid delay of the rock fill. An analysis of the profile on center line of crest and of sections across the south abutment ridge indicate the fill was subjected to the effect of a considerable variation in slope of abutment surface. In addition the depth of fill varied considerably both in longitudinal and transverse section. These factors contributed materially to the uneven rates of settlement.

During early stages of construction the abutments were sluiced to remove loose over-burden prior to the start of rockfill. Abutment excavation and trimming was continued on each abutment above the fill and was generally well in advance of the rockfill except in a section of the right or south abutment above elevation 2230 referred to by the author. Here the amount of abutment excavation necessary to comply with requirements was greater than had been estimated. As a result, the rockfill, which progressed at a rate of approximately 75,000 cubic yards per month, got ahead of the right abutment cleanup. By means of stage construction, with increments of more than one 25 ft. lift underway at one time, delay was avoided and fill lifts were nearing completion at the south abutment contact as successive sections of the abutment stripping met the requirement. In this manner construction delay was avoided and specifications as to height of lift were observed. Even so the piecemeal construction of the fill adjacent to the south abutment, (detail of which is too involved for review in this discussion), resulted in conditions within the fill and packed rock which could only further complicate the settlement problem.

Approval of cutoff alignment and excavation was withheld pending completion and approval of abutment stripping. To illustrate the situation, when the fill and packed rock were above elevation 2375 the cutoff concrete in each abutment was at elevation 2240. Excavation of the cutoff in the reach involved on the south abutment, elevation 2230 to 2350, was on the face of a steep cliff. The bench and trench excavation required was such that the cliff above the cutoff wall was in an unstable position tending to be under-cut by the open trench excavation. As a means to reduce the degree of hazard and to keep pace with the construction of fill and face slabs and avoid possibility of an extensive delay in the completion of the dam, the cutoff trench was filled with concrete with provision for future deepening as necessary to satisfy the requirement of the State Engineer. Also at this time drilling and grouting of cutoff included the unfinished section. Fourteen holes were drilled to depths of 150 feet within the cutoff limits in question and were grouted with a total of 154 sacks of cement.

During 1935 the unfinished section of cutoff wall was completed. The excavation beneath concrete was made by inclined stope and shaft. An adit along the side of the concrete cutoff provided access beneath the previously placed concrete. The inclined tunnel followed beneath the concrete for a distance of approximately 160 feet on slope. Access to the upper end was accomplished by raising to the surface through a shaft some 35 feet along the outer face of the cutoff previously concreted. The work involved about 320 cubic yards of

tunnel excavation to develop access to the unapproved cutoff and to intercept seams in question and remove unsatisfactory rock. The incline and shaft were filled with concrete. Approval of the cutoff in June of 1935 was followed by additional drilling and grouting. A total of 33 holes were drilled and grouted consisting of 27 holes of 7" diameter varying in depth from 100 feet to 80 feet and 7 holes of 4-1/2" diameter and depth 80 feet. The average depth of drilling in rock below concrete was approximately 40 feet. Each hole was grouted in a continuous operation working from the bottom up through a 1" pipe insert with pressure starting at 25 psi and increased to 100 psi at refusal. 785 sacks of cement were used in the grout placed at that time which concluded the cutoff construction.

A change of plan during construction affected the joint between face slab and cutoff. In Figure 4(d) a typical detail of cutoff wall floater slab and face slab connection is illustrated. The writer invites attention to the provision for "small round stone" to be placed under the hinged floater slab. The flexibility intended in this design would quite likely have been greater than actually experienced in the hinged slab as built. The gravel fill material was omitted and a portion of the floater slab was placed over fill consisting of fine quarry run of sizes ranging from chips to 1 cubic foot. Other portions of the floater slab were supported by packed rockfill. This may account in part for failure of the hinged slab to follow the movement in the fill. However, the extent to which the change in fill material beneath the floater slab may have affected the flexibility of the hinged joint would be difficult to evaluate. Also because of the amount of settlement experienced it is considered doubtful that other conditions of fill under the floater slab would have materially changed the extent of damage to the connection between face slabs and cutoff wall.

In conclusion, in the opinion of the writer, the beneficial effect of sluicing rockfill cannot be overstressed. It may also be noted that such an opinion is in essence a restatement of comments in discussion by other writers. (References are those given by the author.) In retrospect it seems evident now that a number of factors contributed to the failure of face slab experienced at Cogswell Dam. The dry fill construction method, admittedly not the only cause of excessive settlement of fill, certainly was the greatest single factor. Even a fill of clean large rock when properly sluiced is given a degree of beneficial lubrication and settlement during construction is aided. Where fines are present sluicing is indispensable. It is believed to have been clearly shown that adequate sluicing of the fill during construction of the dam would have settled the fill at the time of construction to a condition which would have made the construction of a permanent concrete face possible. And as an additional illustration, later experienced at San Gabriel Dam where the dumped fill was sluiced with two parts of water to one rock by volume further demonstrated the beneficial value of sluicing by the amount of initial settlement observed during construction.

Discussion of the cutoff construction problem experienced and its solution is offered in the hope that should a situation of a similar nature occur on other construction it may be of benefit to those involved to know of previous experience. The method used was well justified both through elimination of a dangerous excavation hazard and by the avoidance of construction delay. The adequacy of the cutoff and grouting has been well demonstrated during the years since construction. Other than a blanket of gunite the abutment has required no attention since completion of the dam.

ROCKFILL DAMS: CHERRY VALLEY CENTRAL CORE DAM[a]

Discussion by Joseph C. Dodd

JOSEPH C. DODD,[1] J.M. ASCE.—This paper describes a central core rockfill type dam on a rock foundation having a maximum utilization of core material consistent with minimum total embankment volume. Under these circumstances the core will usually be wider than that necessary to hold seepage within the desired limits, as the width is dictated by material availability and economic factors rather than permeability. The use of a drainage blanket under the core downstream of the grout curtain is frequently advantageous in this situation, as the seepage pore pressures affecting the downstream slope stability may be reduced with the lowering of the phreatic line. The blanket assists in relieving temporary pore pressures in the lower portion of the core developed during construction. Also, a large proportion of the seepage crosses the horizontal filter into the drainage blanket, rather than across the stacked filter, as necessary without a blanket. Placement of the horizontal filter is much easier to control than the stacked filter, where the dangers of contamination and variation in thickness are much greater. A reduction in quantity of filter material may be achieved in spite of the greater area with the blanket included, particularly if the phreatic line does not intercept the stacked filter. The thickness of the horizontal filter may be much thinner than the stacked filter because of the method of placement and improved control. The stacked filter cannot be omitted entirely, but a minimum practical width should provide adequate safety against any small hydraulic gradient across the filter resulting from relief of construction pore pressures or variation from the theoretical phreatic line. This was particularly important in the case of the 167' ± high dam being constructed for the Companhia Urbanizadora da Nova Capital at Brasilia in Brazil, where the filter material had to be produced by crushing and processing quarried rock. It was possible to confine the theoretical phreatic line within the core zone by the use of a blanket, thus requiring a much thinner stacked filter than would have otherwise been necessary. The core was made as wide as possible for the desired stability with 2:1 slopes for the outer rock faces, as the very impervious lateritic core material was much more economical than the quarried rock. A slope of 1:1 for the boundary of the core and rock zone was found to be adequate, considering seepage and anticipated construction pore pressures.

The factors favoring a drainage blanket were much greater at Brasilia than in the case of the Cherry Valley Dam, however, as the permeability of the core material was considerably smaller and hence construction pore

a. Proc. Paper 1733, August, 1958, by H. E. Lloyd, O. L. Moore, and W. F. Getts.

1. Sr. Engr., Sverdrup & Parcel, Inc., San Francisco, Calif.

pressures would dissipate less rapidly, and filter material was much more difficult to obtain. Also, the economics of placement of thin stacked filter zones with greater hand labor would not be as advantageous in this country as elsewhere.

The advisability of a drainage blanket for this type of rockfill dam is, therefore, dependent on many factors peculiar to each situation which must be carefully weighed in terms of cost and performance.

The authors are to be congratulated on the excellent performance of Cherry Valley Dam, as evidenced by the very low values of seepage and settlement during the period since construction.

ROCKFILL DAMS: BROWNLEE SLOPING CORE DAM[a]

Discussion by George K. Leonard

GEORGE K. LEONARD,[1] M. ASCE.—Although the paper gives no hint of what the final unit costs of rock excavation and rock fill will be it is evident, with almost all of the required rock fill being furnished by the required excavation, that the combined cost of the excavation and the rock in place will be unusually low. Also, the estimated kilowatt cost of $175 for the initial installed capacity whould be pleasing to the owners.

The project planning was thorough and complete. Full advantage was taken of all features of site topography and geology, the availability of rock and impervious core material, and knowledge of anticipated weather conditions which would affect work schedules and diversion schemes.

The plan for economical diversion during construction was especially noteworthy. There must be considered risks taken in all diversion plans for it is generally uneconomical to design cofferdams and diversion channels to handle maximum possible flows. Plans made at Brownlee involving flow over the partially completed fill worked out nicely although some fast work by the construction forces was necessary. The planners, designers, and constructers all did a praiseworthy job in completing this fine dam on a fast schedule.

a. Proc. Paper 1734, August, 1958, by Torald Mundal.
1. Chf. Engr., TVA, Knoxville, Tenn.

ROCKFILL DAMS: THE BERSIMIS SLOPING CORE DAMS[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The paper is a valuable contribution to the literature on rockfill dams. The authors have treated the geological aspects of the sites fully, as warranted by the deep overburden on the north abutment of the Bersimis dam.

This deep overburden constitutes, perhaps, the most outstanding feature of the Bersimis dam. The relative grain size of the material contained in the varved clay and silt strata as contrasted with the glacial till is noteworthy. It would be interesting to know if the field investigations turned up any evidence of instability of the material from the strata of varved clay and silt, in view of the behaviour of much material of a somewhat similar nature found throughout the Canadian Shield, which can best be described as being a rock flour. R. F. Legget* has drawn attention to the properties of these materials and indicated their highly impervious nature if consolidated, confirmed also by the authors' finding for the co-efficient of permeability at 10^{-6} cm/sec. Legget has also noted their extreme instability in the presence of water in excess of the liquid limit. In this connection it is worth noting the authors' reference to the "overconsolidated" nature of the overburden on the abutment.

The authors' experience with the seismic method employed for the geophysical survey of the north bank of the Bersimis, with its maximum error of 45 ft. in a 220 ft. depth is to be noted; the geophysical survey indicating the rock surface to be higher than that found by drilling. Similar results have been found with other methods of geophysical surveys and is not peculiar to Canadian conditions, analogous results having been secured in tropical countries. The geophysical methods are useful but must be proven by actual diamond drilling.

The authors refer to the special precautions exercised in dumping rockfill on the north abutment of the Bersimis dam, relating the need for this to the transition materials covering the overburden and the necessity for avoiding displacement of this material by falling rock. The authors would add to the value of the paper if they indicated how the sluicing water was handled.

The usage of calcium chloride for the construction of the impervious zone as late as possible during the autumn season is an interesting confirmation of a procedure which has been adopted elsewhere. Can the authors state if they have found anything detrimental in such a practice?

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a. Proc. Paper 1740, August, 1958, by F. W. Patterson and D. H. MacDonald.

1. Chf. Engr., Power Department, Aluminium Laboratories Ltd., Montreal, Canada.

* "Soil Engineering at Steep Rock Iron Mines, Ontario, Canada." By Robert Ferguson Legget. Paper No. 6304, Proceedings, Institution of Civil Engineers, Vol. II, pp. 169-188, October, 1958.

ROCKFILL DAMS: THE PARADELA DAM-FOUNDATION TREATMENT[a]

Discussion by Judson P. Elston

JUDSON P. ELSTON,[1] M. ASCE.—The paper by Mr. Weyermann was of great interest to the writer. The description of the fault material contained in the granite foundation brings out vividly the tremendous problems that confronted the designers. It would appear that most of the grouting work was conducted in material corresponding to a coarse sand, with generous portions of clay. Mr. Weyermann is to be congratulated upon his excellent presentation of the foundation problems and his detailed thorough description of the steps taken to remedy a very serious situation.

Extreme difficulty has always been experienced in attempting to grout off silty sands and sandy clays not only with cements but with chemical grouts. It many times has been relatively simple to inject water into such materials but because of solids in the mix larger than the voids to be filled or because of colloidal action, viseosity and specific gravities higher than the water itself, it was found next to impossible to do a satisfactory job of sealing off the materials.

The treatment of the gouge material by using both silica gels and cements appears to be a rather novel and interesting approach. The writer has had considerable experience with cements and with chemicals in foundation treatment. The idea of a combination of grouting materials as described by Mr. Weyermann is certainly a valuable contribution to the science of foundation treatment. It is hoped we shall hear more of the individual foundation problems encountered by engineers throughout the world. There is a growing need for such dissemination of engineering data and practices particularly as we are being faced with poorer foundation materials as time goes on.

a. Proc. Paper 1748, August, 1958, by Walter J. Weyermann.

1. Asst. Manager Foundations; Uhl, Hall & Rich, Niagara Falls, New York.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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c. Discussion of several papers, grouped by divisions.

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The Power Division feels a keen responsibility in collecting and making available the latest information on all phases of power engineering, and the Executive Committee will welcome offers of papers or suggestions of subjects which are worthy of being treated in a symposium.

Symposia

The Board of Direction some years ago expressed itself in favor of symposia, particularly for the June and October meetings of each year. Several successful symposia have been arranged by this Division, including those on "Arch Dams," "Rockfill Dams," "Penstocks," "Underground Power Plants" and "Civil Engineering Aspects of Thermal Power Plants." The papers comprising the two first-named are available from Society headquarters in single

volume editions (order form for "Rockfill Dams" attached) and similar volumes covering the other subjects can be issued if their is sufficient demand. If interested, please write to Mr. John F. Bonner and indicate the extent of your interest. In conjunction with the Hydraulic Prime Movers Committee of the ASME, the Division also sponsored a symposium on "Welded Spiral Cases" and copies of the collected papers are available from the ASME. A symposium on "Control of Cracking in Masonry Dam" will be held in the February meeting in Los Angeles, as noted below.

Research

The Society is anxious to promote research on worthy subjects and members are urged to communicate to Mr. Marcel P. Aillery any subjects in which research would be advantageous.

Future Meetings

At the Los Angeles Convention on February 9 - 13, 1959 the Power Division will hold four sessions as follows:

"Cooling Water Supplies for California Steam Electric Stations on Tidewater" by R. W. Spencer and John Bruce, Southern California Edison Company, Los Angeles, California.

"Generation of Power from Geothermal Steam at Geysers Power Plant" by Ben C. Albritton and Albert W. Bruce, Pacific Gas and Electric Company, San Francisco.

"Ocean Cooling Water Systems for the Scattergood and Haynes Plants and Related Construction and Dewatering Problems" by William A. Hunsucker, and L. T. Mariner, Department of Water and Power, City of Los Angeles, California.

SYMPOSIUM SPONSORED BY COMMITTEE ON CONTROL OF CRACKING IN MASONRY DAMS

"Control of Cracking in TVA Masonry Dams" by Walter F. Emmons, Olav Lavik, and Paul L. Hornby, Tennessee Valley Authority, Knoxville, Tennessee.

"High-lift Construction Methods for Mass Concrete" by Dr. Otto Holden, Chief Engineer, The Hydro-Electric Power Commission of Ontario, Toronto, Canada.

"Bureau of Reclamation Practice for Control of Cracking in Arch Dams" by Charles L. Townsend, U. S. Bureau of Reclamation, Denver, Colorado.

"Corps of Engineers Practice and Experience in Controlling Cracking in Masonry Dams" by William R. Waugh and J. A. Rhodes, Office of the Chief of Engineers, Corps of Engineers, Washington, D. C.

"Summary Discussion on Control of Cracking in Masonry Dams" by K. B. Keener, Chairman, Committee on Control of Cracking in Masonry Dams.

"Civil Engineering Aspects of the Santa Susana Experimental Nuclear Power Station" by Dallas I. Downs, Southern California Edison Company, Los Angeles; Robert F. Boggus, and George E. Deegan, North American Aviation, Inc., Canoga Park, California.

"Vallecitos and Humboldt Bay Atomic Power Plants" by F. F. Mautz and Gordon L. Coltrin, Pacific Gas and Electric Company, San Francisco, California.

The Summer Convention 1959 will be held in Cleveland, Ohio on May 4 - 8 and the Power Division expects to present an interesting program. The Annual Convention this year in Washington, D. C. on October 19 - 23 will have as its theme the work of American engineers, contractors, and governmental agencies in other countries. The Power Division program is taking shape and will be announced in a future newsletter.

MONTHLY SCIENTIFIC PUBLICATION OF INTEREST TO POWER DIVISION MEMBERS

It will be of interest to our Division members to know that the American Geophysical Union is combining the scientific and technological material in its bimonthly TRANSACTIONS with the quarterly JOURNAL OF GEOPHYSICAL RESEARCH and will issue the JOURNAL OF GEOPHYSICAL RESEARCH as a monthly starting in 1959. This will contain much material bearing on Hydrology of interest to our members.

The subscription rate for the new JOURNAL is $16.00 for the calendar year. Memberships, however, are invited. Membership is $10.00 per calendar year and includes a subscription to the JOURNAL. Any who are interested should write to the office of the American Geophysical Union, 1515 Massachusetts Avenue, N. W., Washington 5, D. C.

News from the Southeast

Following the Sixth Congress of the International Commission on Large Dams held in New York in September 1958, a portion of this group visited the southeast for a study tour.

This group consisted of about 75 people, 60 of them from other countries, and visited projects of T.V.A. from Bristol to Chattanooga, Hydro Plants of ALCOA in the Smoky Mountain region, Central Alabama Hydro developments and the Corps of Engineers' model basin and concrete laboratories in Vicksburg and Jackson, Mississippi. A few highlights from the visit of the foreign group follow.

These people were, for the most part, from Europe, although Egypt, Japan, Brazil and the West Indies were represented. The group spent a day in Alabama as guests of the Alabama Power Company. They visited a four-state load dispatching center in Birmingham in the morning. The gentlemen in the group, in the afternoon, visited the site of the 300' high Lewis Smith rock-fill dam on which construction had begun and the river diversion tunnel completed. The reception area at the dam was equipped for discussions with samples of materials and large key drawings of the project. This group was interested not only in this project but in our American way of life; how our people got to work; what we did with junk automobiles; type of crops grown; species of timber; areal geology; water supply; population; types of industry; transportation, etc. All the visitors had cameras and many pictures were taken, including local street scenes and a cotton field where the buses stopped and everyone disembarked momentarily. The dams were visited by means of air conditioned buses equipped with PA systems over which the guides kept up a running talk covering the country side, our ways of life and political comments as a result of direct questions from the visitors.

The ladies on the trip had a special local sightseeing tour of the Birmingham area during which they visited a super market, large drug store, dress shop, high school and both white and colored residential sections of the

city before they met for tea in a typical American home. They showed particular amazement at the equipment in a modern American kitchen.

The ladies' tour and the men's tour terminated at a mountain top club overlooking the city of Birmingham, where they all relaxed, met various civic personalities, and had dinner. Later that evening the group left by train for Vicksburg.

Their comments, professionally, indicated that the Europeans are ahead of the U. S. in the technique of hydro-electric power dam building and that this was necessitated by the fact that they were short of funds and had to be economical in the use of materials.

The people in the Alabama area were delighted to entertain and enjoyed immensely this distinguished group of foreign engineers.

(Contributed by R. S. Woodruff.)

Cooperation with Other Societies

The Power Division has for many years maintained liaison with the Hydraulic Prime Movers Committee of the ASME and is represented on that committee. Some recent ASME papers of interest are:

Paper No.	58-A-60	"Symposium on Welded Spiral Cases from the Manufacturer's Viewpoint"
	58-A-76	"Free Discharge through a Turbine Distributor, Case and Draft Tube"
	58-A-100	"Some Hydraulic Features of Punledge Generating Plant"
	58-A-108	"Riversible Pump-Turbines for Sir Adam Beck—Niagara Pumping—Generating Station"

These papers are available from ASME headquarters.

The Power Division is also represented on the Policy Planning Committee of the American Power Conference, which is held in Chicago, Illinois in the spring of each year.

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Robert A. Sutherland
Newsletter Editor

American Society of Civil Engineers
33 West 39th Street
New York 18, New York

Please send me _____ copy(s) of the Symposium on Rockfill Dams (Proc. Symp. Series No. 3) at a list price of $8.00 each (50% discount to members of ASCE).
(1) Enclosed is my remittance. Grade _____

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